APRIL 1956

Journal

VOL. 48 • NO. 4

AMERICAN WATER WORKS ASSOCIATION

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Journal

AMERICAN WATER WORKS ASSOCIATION

2 PARK AVE., NEW YORK 16, N.Y. • Phone: MUrray Hill 4-6686

April 1956

Vol. 48 • No. 4

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Journal AWWA is published monthly at Prince & Lemon Sts., Lancaster, Pa., by the Am. Water Works Assn., Inc., 2 Park Ave., New York 16, N.Y., and entered as second class matter Jan. 23, 1943, at the Post Office at Lancaster, Pa., under the act of Aug. 24, 1912. Accepted for mailing at a special rate of postage provided for in paragraph (d-2), Section 34.40, P. L. & R. of 1948. Authorized Aug. 6, 1918. $7.00 of members' dues are applied as a subscription to the Journal; additional single copies to members—60 cents; single copies to non-members—85 cents. Indexed annually in December; and regularly by *Industrial Arts Index* and *Engineering Index*. Microfilm edition (for Journal subscribers only) by University Microfilms, Ann Arbor, Mich.

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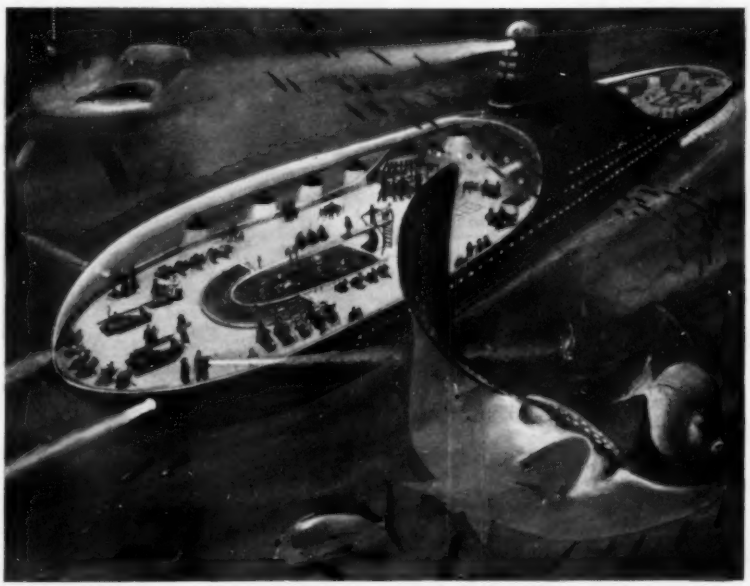

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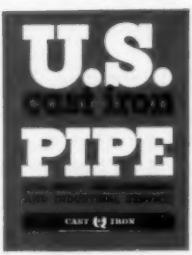

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St. Louis, Mo. **May 6-11, 1956**

All reservations will be cleared through the AWWA office. The nine official hotels have agreed to accept no reservations for the 1956 Conference except as they are requested on the standard form, through the AWWA.

Coming Meetings

AWWA SECTIONS

Apr. 18–20—New York Section, at Hotel Utica, Utica. Secretary, Kimball Blanchard, Rensselaer Valve Co., 11 W. 42nd St., New York 36.

Apr. 23–25—Canadian Section, at Hotel London, London, Ont. Secretary, A. E. Berry, 72 Grenville St., Toronto, Ont.

Apr. 26–28—Pacific Northwest Section, at Empress Hotel, Victoria, B.C. Secretary, F. D. Jones, Room 305, City Hall, Spokane, Wash.

Jun. 5—New Jersey Section Summer Outing. Secretary, C. B. Tygert, Wallace & Tiernan Inc., Box 178, Newark 1.

Sep. 12–14—New York Section, at Sagamore Inn, Bolton Landing, Lake George. Secretary, Kimball Blanchard, Rensselaer Valve Co., c/o Ludlow Valve Co., 11 W. 42nd St., New York 17.

Sep. 12–14—Michigan Section, Kalamazoo. Secretary, T. L. Vander Velde, Chief, Water Supply Section, Michigan Dept. of Health, Lansing 4.

Sep. 12–14—North Central Section, at Hotel Lowry, St. Paul, Minn. Secretary, Leonard N. Thompson, Gen. Mgr., Water Dept., St. Paul 2, Minn.

Sep. 17–19—Kentucky-Tennessee Section, at Hotel Patten, Chattanooga, Tenn. Secretary, J. W. Finney Jr., 553 S. Limestone St., Lexington, Ky.

Sep. 19–21—Ohio Section, at Commodore Perry Hotel, Toledo. Secretary, M. E. Druley, Dayton Power & Light Co., Wilmington.

Sep. 26–28—Wisconsin Section, at Stoddard Hotel, La Crosse. Secretary, L. A. Smith, Supt., Water & Sewerage, City Hall, Madison 3.

Sep. 30–Oct. 2—Missouri Section, at Hotel Governor, Jefferson City. Secretary, W. A. Kramer, Rm. 3, 6th

(Continued on page 8)

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WITH THE **RING-TITE** COUPLING

Coming Meetings

(Continued from page 6)

Floor, State Office Bldg., Jefferson City.

Oct. 14–17—Southwest Section, at Marion Hotel, Little Rock, Ark. Secretary, Leslie A. Jackson, Mgr.-Engr., Municipal Water Works, Robinson Memorial Auditorium, Little Rock, Ark.

Oct. 18–20—New Jersey Section, at Hotel Madison, Atlantic City. Secretary, C. B. Tygert, Wallace & Tiernan Inc., Box 178, Newark 1.

Oct. 21–24—Alabama-Mississippi Section, at Battle House, Mobile, Ala. Secretary, Irving E. Anderson, Dist. Engr., Surface Water Branch, USGS, Box 2052, Jackson, Miss.

Oct. 22–26—California Section, at U. S. Grant Hotel, San Diego. Secretary, Henry J. Ongerth, Sr. San. Engr., Bureau of San. Eng., 905 Contra Costa Ave., Berkeley 7.

OTHER ORGANIZATIONS

Apr. 17–18—Annual Municipal and Sanitary Engineering Conference, University of Florida, Gainesville, Fla.

Apr. 26–27—Annual Management Engineering Conference, sponsored by Society for Advancement of Management and ASME Management Div., at Hotel Statler, New York. Society for Advancement of Management, 74—5th Ave., New York 11.

May 4—Annual Conference for Engineers, College of Engineering, Ohio State University, at Ohio Union, Columbus.

May 12–15—German Engineers Assn. Centenary Celebration. Verein Deutscher Ingenieure, Prinz-Georg-Strasse 77/79, Dusseldorf, Germany.

May 14–15—Annual Meeting, Society of American Military Engineers, Washington, D.C.

May 23–26—Annual Meeting, National Society of Professional Engineers, at Ambassador Hotel, Atlantic City, N.J.

Jun. 4–8—National Fire Protection Assn. Annual Meeting, Hotel Statler, Boston, Mass.

Jun. 4–8—American Society of Civil Engineers, Knoxville, Tenn.

Jun. 17–23—World Power Conference, Vienna, Austria. US National Committee, World Power Conference, 29 W. 39th St., New York 18, N.Y.

Jul. 30–Aug. 3—Gordon Research Conference on Ion Exchange, American Assn. for Advancement of Science, at Kimball Union Academy, Meriden, N.H. W. George Parks, Director, Dept. of Chemistry, University of Rhode Island, Kingston, R.I.

Oct. 8–11—Federation of Sewage & Industrial Wastes Assns., Statler Hotel, Los Angeles, Calif.

Oct. 14–17—National Institute of Governmental Purchasing, at Conrad Hilton Hotel, Chicago. A. H. Hall, Exec. Vice-Pres., 1001 Connecticut Ave., N.W., Washington 6, D.C.

Oct. 15–17—Annual Convention, American Gas Assn., Atlantic City, N.J. For reservations, write American Gas Assn. Housing Bureau, 16 Central Pier, Atlantic City, N.J.

Oct. 15–19—American Society of Civil Engineers, Pittsburgh, Pa.

Nov. 12–16—American Public Health Assn., Atlantic City, N.J.

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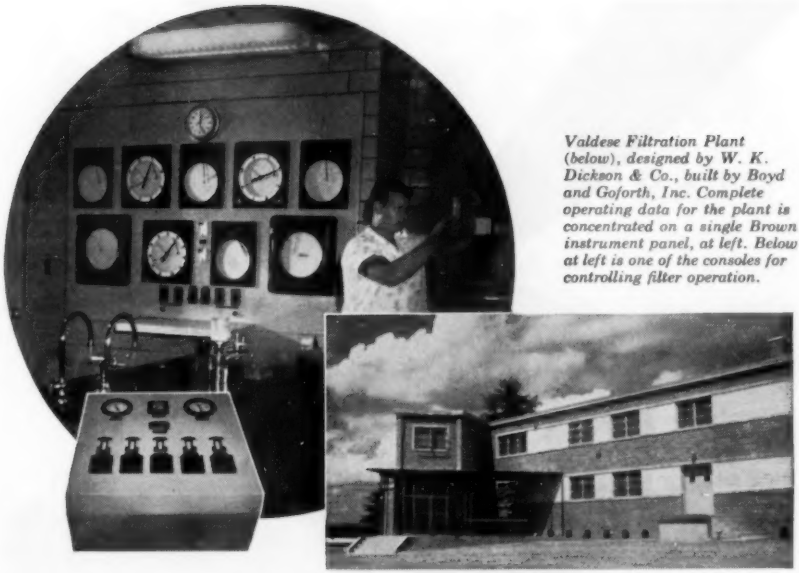

Valdese Filtration Plant (below), designed by W. K. Dickson & Co., built by Boyd and Goforth, Inc. Complete operating data for the plant is concentrated on a single Brown instrument panel, at left. Below at left is one of the consoles for controlling filter operation.

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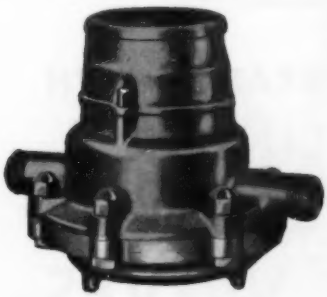
Because it's SO ACCURATE

"American Meters assure full revenue for all the water we pump because they register so accurately, even on the smallest flow. That's why, as Superintendent, I feel safe when I buy American."



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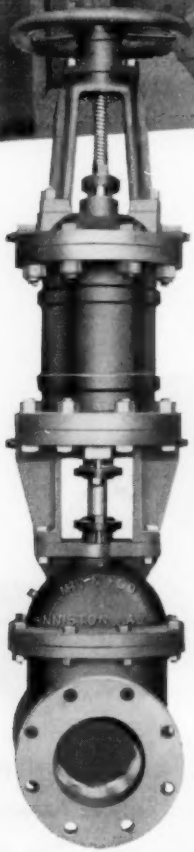

EASY OPERATION WITH

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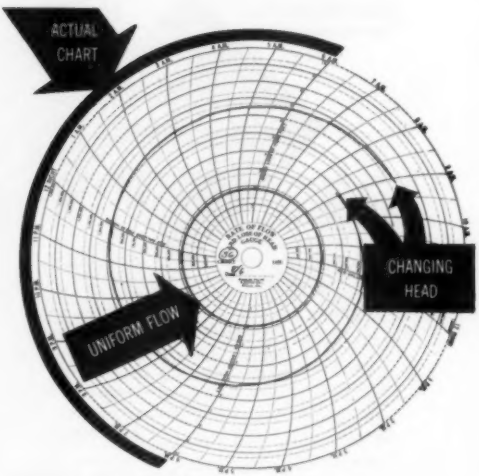
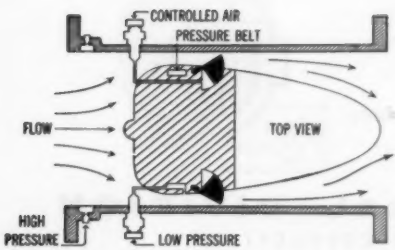
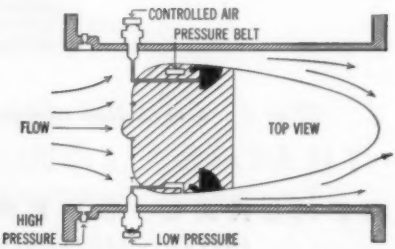
Amazing Modulair® Controller responds fast...closes tight



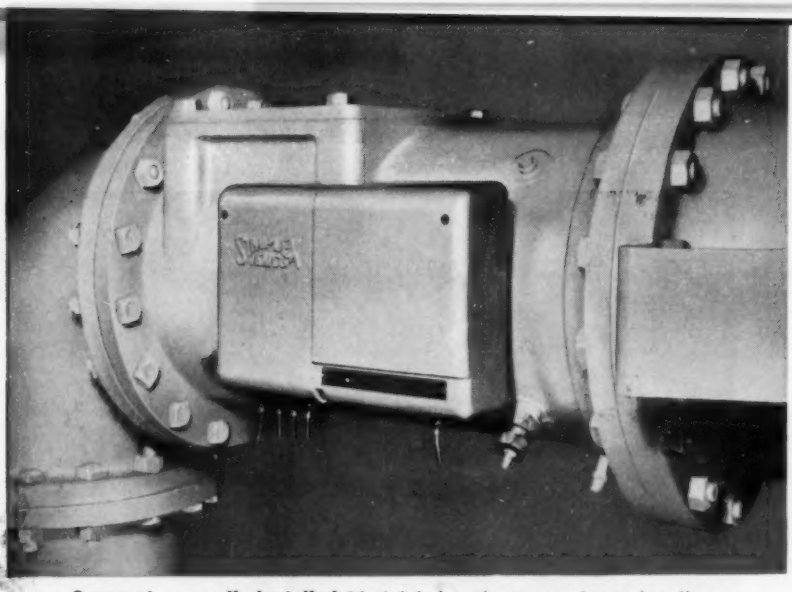
Here's proof that the new Simplex Modulair Filter Rate Controller responds instantly to changes in filter head, holds flow at desired rate...automatically! In chart from Modulair installation No. 2, head changes from 4′ to 7′ while rate forms almost perfect circle at the set 275,000 gpd.



How Modulair works! Pipe-like casting with annular Venturi throat houses streamlined inner body with corded-rubber modulator (black) that expands and retracts pneumatically to control flow. When head is low at start of run, modulator expands automatically to hold flow at set rate.



As filter head loss rises, flow tends to decrease . . . and change the Venturi differential. Sensitive Modulair compensates by instantly retracting the pneumatic modulator, maintaining a constant head-loss value through the combined filter and controller, automatically keeping flow at set rate!



Compact . . . easily installed Modulair is only two to three pipe diameters long! It fits wherever a piece of pipe can fit . . . in any position, horizontal or vertical . . . simplifying pipe gallery layout. High-grade, cast iron Modulair comes in 4" to 24" standard pipe sizes with corrosion-resisting fittings. Cord-inserted rubber throttling element that's as tough as a truck tire has given 30 years' wear in accelerated life-tests.

Accuracy! In 4 years of field performance, Modulair charts show an accuracy in filter control that exceeds today's most exacting demands. This high degree of control extends for wide ranges of head loss and flow rates, including complete, tight shut-off! And there's no drift at end of run.

Longer filter runs! Modulair combines a low head-loss, annular Venturi measuring element with a streamlined throttling element. This new design offers so little resistance to flow that you get longer filter runs, big savings in wash water!

Minimize maintenance! With Modulair's new design, there's no place for air to accumulate . . . no place for sand to lodge and interfere with accuracy. Throttling element, the only moving part inside pipeline, is non-corrosive rubber. Pneumatic poise element is simple, rugged. There are no line valves, no pilot valves, no pistons, no valve shafts, no stuffing boxes to pack!

Lower first cost! Modulair's low head loss means lower operating head requirements . . . permits vertical compression in plant design and lower costs in plant construction.

Completely pneumatic! No intermediate hydraulic steps! No pilot valves! No hydraulic cylinders! Air loading pressure is applied *directly* to the flow controlling element.

Automatic filter operation! Modulair works with control systems of any type to give you remote rate control or master pneumatic control that's fully automatic.

Technical bulletin! For full details on Modulair, write for Bulletin 950. Simplex Valve & Meter Company, Dept. JA-4, 7 East Orange Street, Lancaster, Penna.

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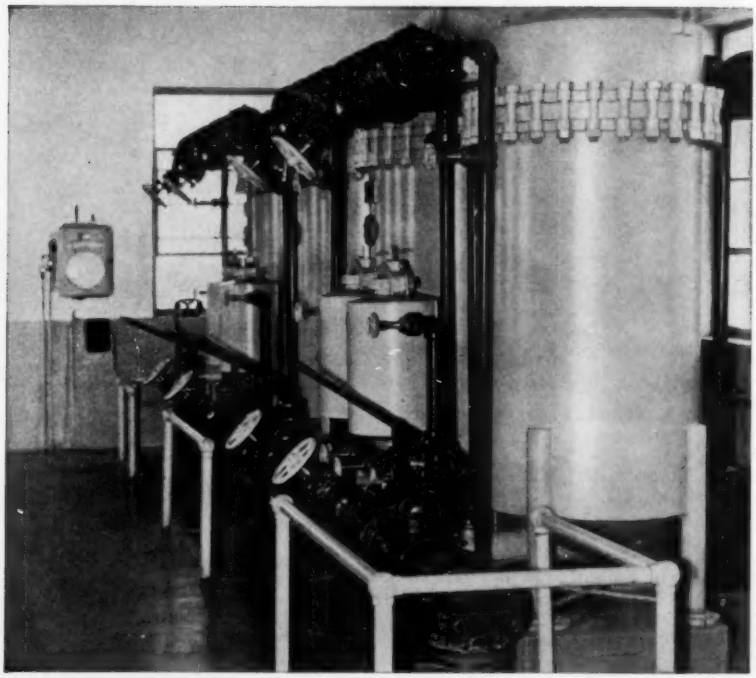

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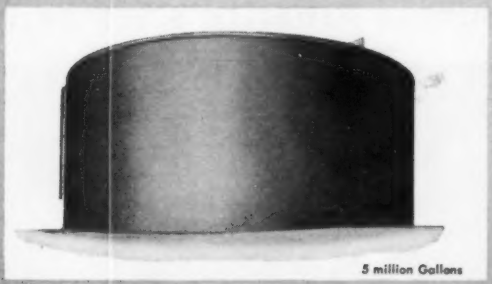

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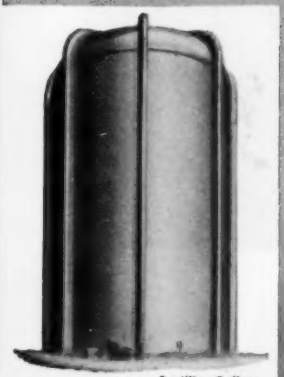

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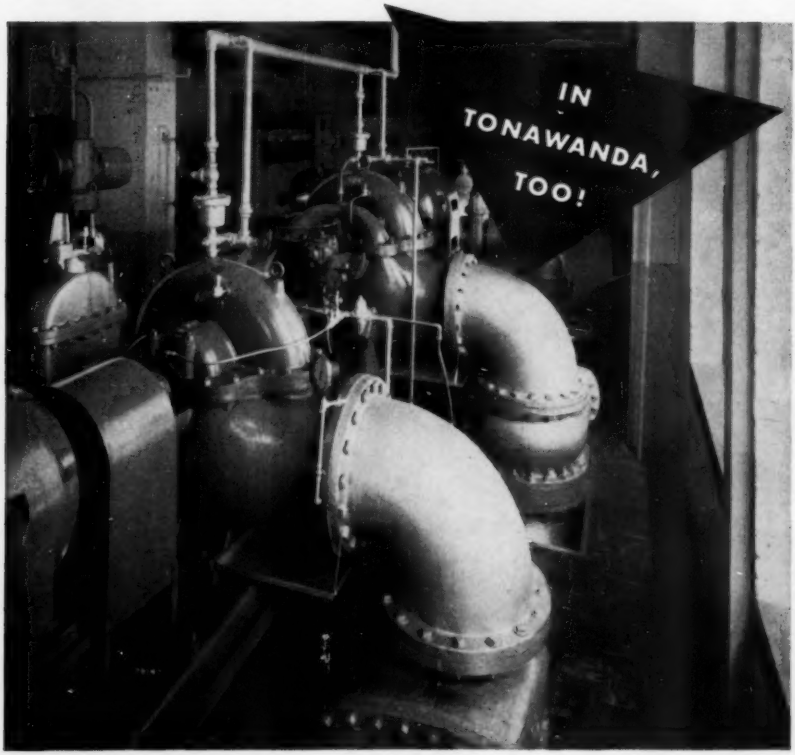

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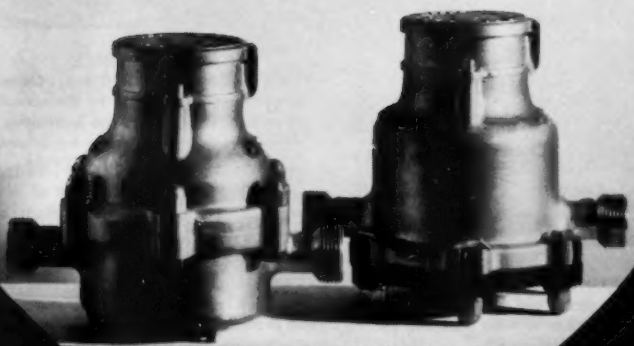

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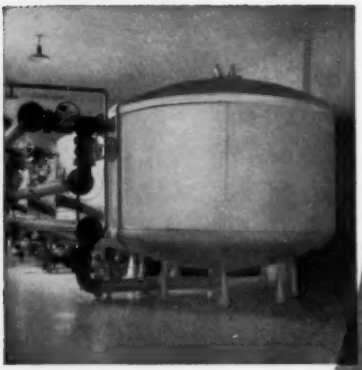
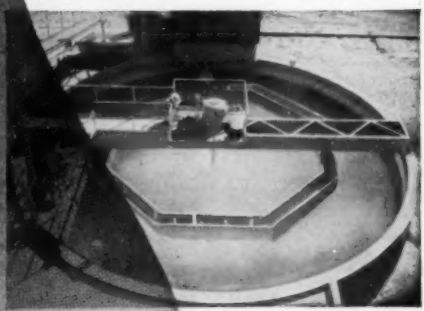


There are many reasons why municipalities and industries have installed General Filter water treating plants:



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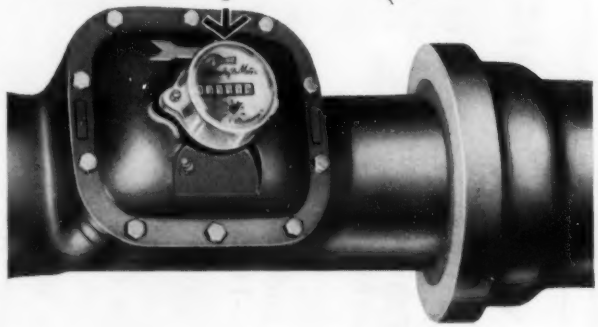

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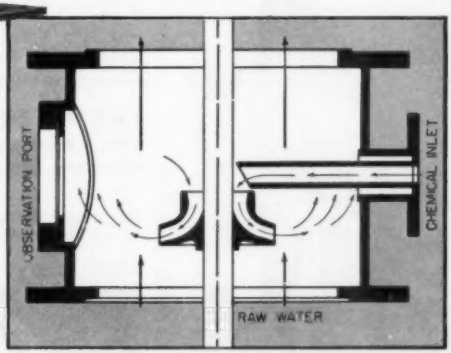

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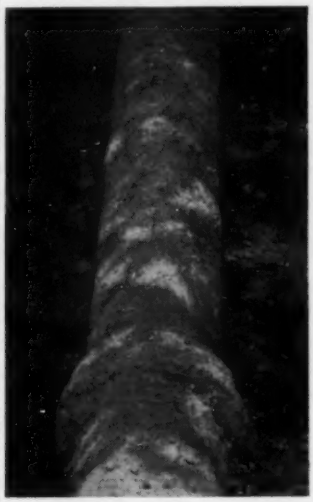

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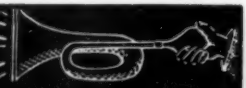

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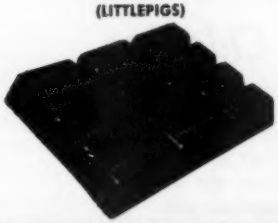

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AMERICAN WATER WORKS ASSOCIATION

VOL. 48 • APRIL 1956 • NO. 4

Controlled Draft From Reservoirs

Panel Discussion

A panel discussion presented on Jun. 15, 1955, at the Annual Conference, Chicago, Ill.

Surface and Underground Storage—*Raymond A. Hill*

A revision of a paper originally presented on Oct. 29, 1954, at the California Section Meeting, Long Beach, Calif., by Raymond A. Hill, Partner, Leeds, Hill and Jewett, Cons. Engrs., Los Angeles, Calif.

FUNDAMENTALLY, storage is a source of supply. It follows that the water in storage must be available when any need arises and in sufficient quantity to satisfy all needs. The functions of storage are limited to providing reserves for emergencies, and equating differences between rates of production and use.

Regulatory storage required to satisfy the second function may be subdivided into long-term hold-over storage, seasonal storage, and storage for equating short-term variations, which usually last a few days.

In approaching the problem of storage, it is necessary to distinguish between emergency storage, from which water may have to be withdrawn during periods of interruption or for abnormal demands, and regulatory storage, from which water is withdrawn to meet normal variations in either the supply or use of water.

Location

The requirement that water in storage be available as a source of supply places some limitations on the location of such storage. The ideal situation, of course, would be to have near the center of use a large surface reservoir that is supplied by gravity and from which water can be withdrawn by gravity. Unfortunately, such an arrangement is seldom possible in practice.

The necessity of elevation limits the location of storage to three possibilities: [1] in elevated tanks above the ground;

[2] in reservoirs or tanks on the ground surface; and [3] in underground wells from which water is withdrawn by pumping.

Where elevated tanks can be used, they have the advantage of supplying water at desired pressures, and in this regard they approach the ideal for emergency storage. This advantage, however, is not needed to satisfy the function of regulation.

In some fortunate communities, surface storage can be provided at elevations high enough to permit gravity withdrawal of sufficient water to meet all demands. Where topographic conditions are thus favorable, both emergency and regulatory supplies can be stored in ground level reservoirs.

Where elevated tanks are prohibited, or where ground level storage cannot be provided at sufficient elevation to permit gravity flow withdrawal at adequate pressures, duplicate pumping facilities or alternate sources of supply are required, even for emergency use. In such cases, storage at ground level becomes merely one of the possible sources from which water can be pumped into the distribution system.

Underground storage serves the same function as surface storage which is below the pressure gradient in that water may be pumped from wells and delivered into the distribution system in the same manner and to the same extent that water is pumped from a surface reservoir to meet emergencies and variations in water use. Seldom, however, is full advantage taken of the opportunity to use underground storage for regulation of short-term variations in demand.

Amount Required

In any system, the actual amount of storage that should be available for both emergency and regulatory use depends upon the characteristics peculiar to that system. In principle, however, this quantity is limited to:

1. Emergency storage in an amount equal to the maximum emergency demand minus the volume available from other sources during the emergency period

2. Regulatory storage in an amount equal to the maximum difference between the accumulated demand and the accumulated production within the period for which regulation must be provided.

The required quantity of emergency storage thus depends upon the abnormal demands which may have to be met, the number of supply sources, and the probability of interruption of one or all of these sources. Such interruptions may be of short duration, as in the case of power outages, or they may extend over relatively long periods of time, as in the case of major damage to an aqueduct bringing water from a remote source. Emergency storage, in other words, is a reserve that should be maintained in sufficient volume to satisfy those demands which cannot temporarily be met from other sources. Such reserves can be small in the case of most systems, but they must be relatively large where water is brought to a community from a great distance.

Most cities and utilities are not concerned with hold-over storage because they can obtain enough water from their sources of supply to meet at least the average annual demand. Storage for regulation of supplies over a period of several years is frequently needed, however, where there is dependence on the runoff of streams.

The need for seasonal regulation in surface reservoirs is also the exception rather than the rule, because the rate of withdrawal from available sources can usually be varied to conform with seasonal changes in the rate of demand. Such storage is customary, however,

where water is brought over long distances to the place of use, because any aqueduct should be operated at substantially full capacity throughout the year.

Regulatory storage for equating short-term variations is required in most water systems, but it does not follow that such storage need be in the form of elevated tanks or surface reservoirs. It is only necessary that this storage be available to meet variations in demand for, at most, a few days. The quantity of regulatory storage required for this purpose is extremely sensitive to the rate at which water can be produced. The peak demand for water on any day is seldom more than twice the average demand for the same day. Fire flows (the chief exception to this rule) are drafts on emergency, rather than regulatory, storage, so that, if water could be produced and delivered at rates double the average demand for any day, there would be no need for regulatory storage. On the other hand, if the quantity of water produced during a day were no more at any time than the average demand for that day, then water in an amount equal to at least one-quarter of the total use on that day would have to be withdrawn from storage to meet demands above the average.

TABLE 1

Relation Between Supply Rate and Regulatory Storage for 24-hr Period

Max. Rate of Supply—*per cent 24-hr avg.*	Required Storage—*per cent total use*
80	50–60
90	35–40
100	25–30
125	10–15
150	0–5

The normal relation between the quantity of storage required for daily regulation and the maximum rate of supply is demonstrated in Table 1.

Obviously, if the rate of production is less than the average demand on one day, the draft on storage becomes accumulative. The values shown in Table 1 reflect this situation over a period of a few days: if regulation over longer periods were required, greater amounts of storage, and eventually a draft on seasonal storage, would be necessary. The essential point is that an increase in supply above the average, whenever it can be used or impounded, reduces the required regulatory storage at a greater rate. If, for example, the use of water on a typical day were 10.0 mil gal and water were delivered to one system at a uniform rate equal to this demand, then at least 2.5 mil gal of regulatory storage would be needed; but if water were delivered into the same system at 12.5 mgd, whenever it could be used or stored, the draft on storage would be reduced by at least one-half.

Use of Underground Storage

The relationship described, between the rate of production and regulatory storage requirements, may be used to advantage by varying the production rate of wells located in or near the service area. Any ground water basin, after all, is a natural reservoir; and, from an operating standpoint, there is no difference between a deep well pump and a booster pump at a surface reservoir—both draw upon storage as a source of supply.

Except in the case of large water systems, it is usually impracticable to provide sufficient surface storage for seasonal regulation. In those systems which are supplied wholly or largely by ground water sources, however,

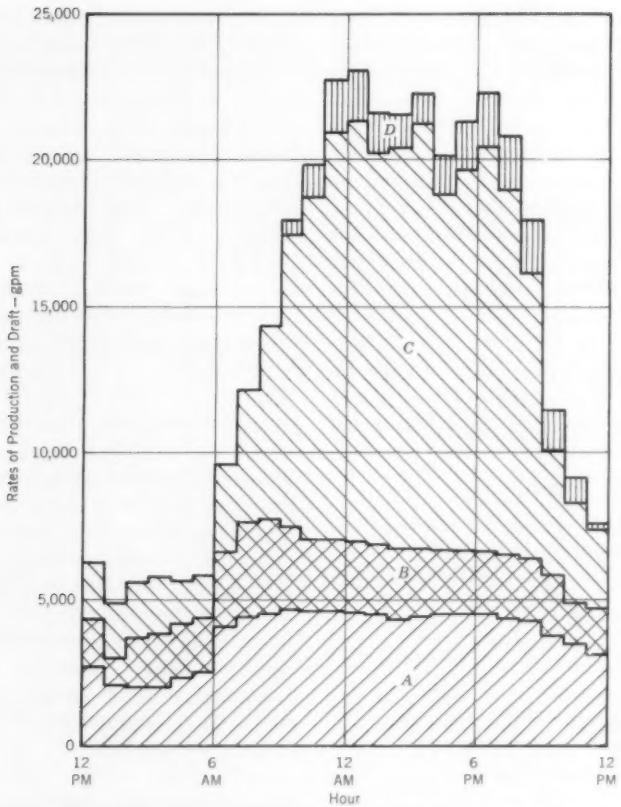

Fig. 1. Sources of Supply on Typical Summer Day, West Coastal Basin

Area A *represents water purchased from metropolitan supply;* B, *water piped from distant wells;* C, *the draft on local underground storage; and* D, *the draft on surface storage. The rates of flow are in average gallons per minute for each hour.*

such storage—and even hold-over storage—is available. Underground storage thus affords regulation which would not otherwise be possible, or would at least be much more expensive.

The principal disadvantage of underground storage is that water can be obtained from it only by pumping. The supply, therefore, is subject to interruption and cannot meet emergency storage requirements as well as tanks or surface reservoirs from which water can be withdrawn by gravity. Frequently, however, where elevated tanks cannot be used and where the topography does not permit surface storage at high elevations, underground storage is better suited to emergency needs than surface storage reservoirs from which water must be pumped. This is because wells are generally distributed over a wide area, while surface storage

reservoirs and booster plants are usually concentrated in one or two places. The risk of interruption of the supply from ground water sources can thus be minimized more readily. Furthermore, reserve units consisting of a well and a pumping plant can be provided at comparatively small cost.

There are, of course, instances of legal restrictions upon the use of ground waters, but such restrictions are usually based upon the total annual draft rather than pumping rates. In the west coastal basin bordering on Santa Monica Bay, for example, the total annual draft upon ground waters is soon to be restricted. Most of the dependent municipalities whose well production must be curtailed can obtain enough water (either directly or indirectly) from the Metropolitan Water District of Southern California to provide the difference between the limited well production and the total production required to satisfy their individual demands.

Because of this limitation upon the total annual draft, and because the supply from the metropolitan district must be drawn at reasonably constant rates, the underground storage supply will have to be used for seasonal regulation. The use of ground waters will thus be small in the winter months, compensating for the summer pumpage in the total annual figures.

These same ground waters can be used to equate daily variations in demand and have been so used by one of the principal utilities in that area. For example, although during successive hot days in August 1954 this utility produced or purchased a total of 19.8 mil gal of water on the first day, 20.6 mil gal on the second, and 21.8 mil gal on the third, it was able, by effective use of the wells on its system, to limit the net draft upon surface storage, from the high level on the first day to the minimum level on the third day, to 1.7 mil gal. During these three days of high demand, 26 per cent of the water was obtained from the Metropolitan Water District and 16 per cent imported to the area from wells located at a considerable distance. The remaining 58 per cent was produced from wells within, or reasonably near, the service area of that utility. The water obtained from the Metropolitan Water District and that produced from wells outside the area was delivered to the system as fairly uniform rates. The rate of production from the local wells, on the other hand, was varied from less than 2,000 gpm to more than 14,000 gpm, generally in conformity with the demands of users.

Typical variations for one of these three days are shown in Fig. 1. As shown in this graph (prepared from actual records of operation) the local ground water basin was used as a storage reservoir. Water was pumped from wells—that is, withdrawn from underground storage—almost as needed to meet variations in demand.

Summary

Fundamentally, storage is merely a source of supply from which water may be drawn when needed for either of two distinct purposes: first, to meet abnormal demands or other emergencies; second, to equate variations in demand. It is desirable, but not necessary, that emergency storage be in tanks or reservoirs from which water will flow by gravity to the place of use. It is convenient, but by no means necessary, that regulatory storage be so situated.

Underground storage, from which water is obtained by pumping from wells, serves all the functions of surface storage below the pressure gradi-

ent. Such underground storage automatically equalizes variations in supply; it can be used to advantage for seasonal regulation; and it is particularly useful for equalizing daily variations in water use. The production of water from wells should, in fact, be regarded as a draft on storage.

—Limitary Curve—*Angus D. Henderson and A. S. Toth*—

A paper presented by Angus D. Henderson, Cons. Engr., Westbury, N.Y., and A. S. Toth, San. Engr., Hydrotechnic Corp., New York, N.Y.

Water supply reservoirs may be divided into two general categories. There are those having storage capacity for only seasonal runoff deficiencies, which normally refill over a 12-month cycle, and those having storage capacity for annual runoff deficiencies, which normally refill in periods extending over 2 years or longer.

In terms of the purpose for which they are developed and used, reservoirs fall into other classifications. The municipal water supply is designed to deliver a fixed, dependable yield during a period of specified drought conditions; at all other times there is little demand upon its surplus stored water. Distinct from this type of development is the reservoir requiring flexible operation to meet fluctuating demands. The majority of industrial reservoirs are designed for seasonal storage and are operated to meet widely varying demands imposed by either production schedules or plant operations. To successfully operate such reservoirs, a definite plan must be established to keep the operator constantly informed of the conditions of withdrawal and reserve. The method of controlled draft provides a simple means for obtaining maximum use of reservoir storage year after year, and may be worked out for any reservoir or system of reservoirs.

Plotting the Curve

Whether the problem is the investigation of a proposed reservoir or the evaluation of an existing one, the initial task is that of gathering stream flow data for the site. The runoff data should be of long-term reference, covering at least 20 years. It is sometimes necessary to work with data for a nearby stream whose characteristic behavior is similar to that of the one being studied. Once these basic data are assembled in terms of monthly mean flow for the period recorded, a mass curve of runoff may be prepared. The mass curve will indicate periods of low runoff, which will in turn require more detailed study.

The most severe dry period can be discovered by drawing draft lines on the mass curve, using a constant storage volume. Having thus determined the critical runoff period, it is best to translate the flow data into mean values for weekly periods throughout the cycle. The constant dependable draft during the critical period, which has been determined from the weekly mass curve, may now be used to calculate the progressive depletion and refilling of reservoir storage. The plotting of the residual, stored-water volumes throughout the test period may be termed the "limitary curve."

The limitary curve, as shown in Fig. 2, indicates, at any time, the minimum reservoir storage, when draft should be limited to the lowest safe rate of withdrawal, or the dependable yield of the development. By superimposing the limitary curve on a curve of current reservoir operation under runoff conditions more favorable than those of the critical period, any excess storage held in the reservoir becomes apparent.

3. The difference in storage volume between the operations curve and the limitary curve at any given time represents the additional storage available, which may be drawn at rates in excess of the dependable-yield rate. At such times, draft may be increased to a higher rate so that the current-operations curve may approach more closely the limitary curve.

4. If the two curves meet, the rate of draft should be limited to the minimum allowable draft.

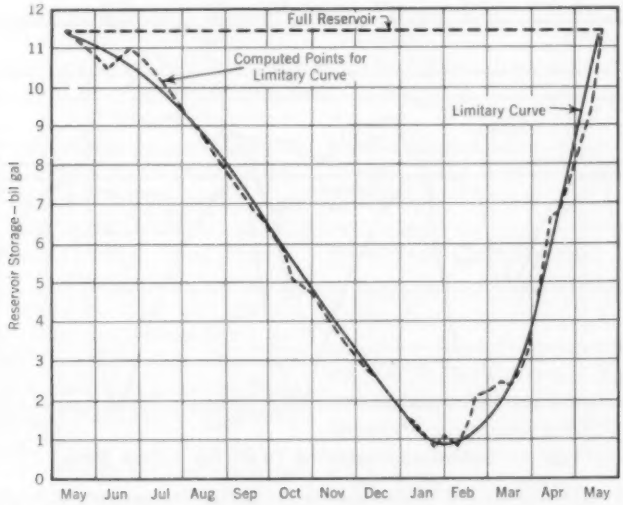

Fig. 2. Limitary Curve of Dependable Yield

Based upon long-term records of critical runoff periods, the curve shows minimum reservoir storage for 12-month period.

Practical Application

The practical application of the limitary curve to the method of controlled draft from a reservoir is relatively simple. The procedure may be outlined in four steps:

1. The limitary curve is plotted as a cyclic curve on graph paper, using units of time for the abscissa scale.

2. The current-operations curve, showing available storage, is plotted on the limitary curve graph.

An illustration of an actual operation is shown in Fig. 3.

The application of this method will result in the full utilization of available storage year after year and will assure the constant availability of a supply equal to the minimum allowable draft during dry-weather periods. Slight deviations of the current-operations curve below the limitary curve during the

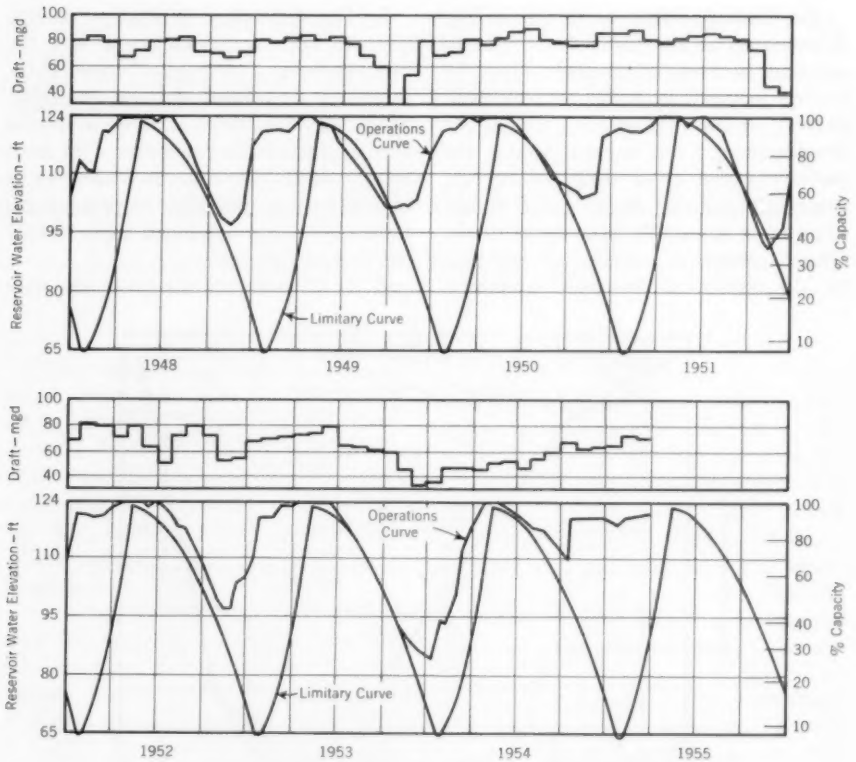

Fig. 3. Record of Controlled Draft for 8-Year Period

The records, complete to the second quarter of 1955, demonstrate the application of the limitary curve to the method of controlled draft. The difference between the limitary curve and the operations curve at any point represents additional storage available at that time.

early depletion part of the cycle may be tolerated for a few weeks to meet special water needs. It is essential, however, that some curtailment in water use be made soon after such heavy drafts so that the balance of storage will return to the favorable position indicated by the limitary curve.

Other Uses

The limitary-curve technique of controlled draft may also be used effectively when the volume of water required is closely related to water temperature, as is the case in all cooling processes. By considering the expected water temperatures of the reservoir supply for the entire year, it is possible to proportion the need for water to achieve maximum effective use of the available supply. Thus, for example, some water may be conserved during the months of May, June, and July, to be drawn for more beneficial application during August and Sep-

tember, when water temperature is normally highest.

The method of controlled draft based on the limitary curve may be applied to a system of reservoirs as well as to individual storage units. In this case it is necessary to obtain the limiting conditions of maximum depletion of the entire system by combining the performance of the individual reservoirs for all unusual dry periods. This method has been used successfully for large integrated water supply systems.

The principle of limitary-storage conditions may also be used in the problem of stream regulation, when it is desired to use the entire contents of the regulating reservoir each season to obtain the highest possible regulated downstream flow.

Impounding Reservoirs—*Edward J. Clark*

A paper presented by Edward J. Clark, Chief Engr., Bureau of Water Supply, Dept. of Water Supply, Gas, & Electricity, New York, N.Y.

The flow of the surface streams from which New York City obtains practically all of its water supply varies from an almost negligible quantity in the summer season to heavy flood flows in the spring. The amount of water required for municipal purposes greatly exceeds the minimum stream flows and impounding reservoirs are therefore necessary to collect and store the flood flows during wet seasons for use during future dry periods.

There is a definite relation between the amount of storage provided in an impounding reservoir on any stream and the maximum amount of water that can be obtained from the reservoir without emptying it. Over a period of many years, there may be abundant stream flow and little need for a large impounding reservoir, but drought periods come without warning, and when they do, the entire capacity of the reservoir may be needed for maintaining a continuous supply of water.

A drought period may be regarded as the weak link in a chain, in that it fixes the maximum amount of water that can be depended upon from the reservoir. The maximum supply of water that can be drawn uniformly from an impounding reservoir through drought periods is called the "dependable supply."

New York City System

Before considering the method used to determine the dependable supply of the New York City sources and the manner in which drafts from the reservoirs are regulated to insure the most satisfactory utilization of these sources, it is necessary to describe briefly the New York City system. This system represents a cost of over a billion dollars and produces a water of excellent sanitary quality. As presently developed, it can safely produce 1,590 mgd during drought periods.

The city, at present, has available the following municipal sources of supply:

1. The Croton River watershed in Putnam and Westchester counties, where water is collected in twelve reservoirs and six controlled lakes, is approximately 25 miles north of the city line. It has an area of 375 sq miles, and possesses an aggregate storage capacity of 103 bil gal.

2. The watersheds of Schoharie and Esopus creeks in the Catskills, where

water is impounded in the Schoharie and Ashokan Reservoirs, respectively, are approximately 100 miles from the city, drain an area of 571 sq miles, and have a reservoir storage capacity of 150 bil gal.

3. The Rondout watershed in the Catskills, where water is impounded in Rondout Reservoir, is about 85 miles from the city, has an area of 95 sq miles, and possesses a reservoir capacity of 50 bil gal. It is part of the current Delaware system, which also includes reservoirs on the Neversink River and the East Branch (Delaware), both tributaries of the Delaware River. The Neversink Reservoir drains an area of 93 sq miles and has a capacity of 35 bil gal. Pepacton Reservoir, on the East Branch, has a capacity of 143 bil gal and is supplied from a watershed area of 372 sq miles.

4. The Ridgewood system on the southern side of Long Island, where water is obtained from ponds, infiltration galleries and driven wells, has a drainage area of about 168 sq miles and extends from the borough of Brooklyn to the Suffolk County line.

5. Wells in Queens and Richmond counties are used locally, as needed, to maintain adequate pressures and to supplement the Catskill supply.

In addition to these municipal sources of supply, there are two private water companies, (the Jamaica Water Supply Company and the New York Water Service Corporation) which furnish approximately 40 mgd to some 450,000 residents in Queens.

In normal operation, the following facts are considered in determining the order in which the sources are to be utilized:

1. The Catskill and Delaware systems are sources of high-level supplies which require no pumping in the city, except for a small quantity which is boosted to some areas above the gradient of the distribution reservoirs.

2. Croton water is delivered to the boroughs of Manhattan and the Bronx by gravity but about two-thirds of this water must be pumped within the borough limits to reach higher ground elevations.

3. Water from Long Island must be pumped: in some cases, the same water is pumped at three separate points.

4. The quality of the Croton and high-level supply waters is much superior to that of Long Island water.

It thus becomes apparent that in time of plenty, for reasons of economy, the high-level supplies are utilized to greatest possible advantage. These are supplemented with Croton water as required to meet the demand. When drought conditions obtain, supplies from the Long Island system and borough sources are used to relieve the drain on the reservoirs.

Dependable Supplies

If runoff records covering one or more drought periods for a given catchment area are available, and the storage capacity of the impounding reservoir is fixed, the dependable supply can be determined by either the mass diagram method or Hazen's probability method. The mass diagram method is reliable only when runoff records are available for a considerable period of time—at least 25 years. If records for fewer than 25 years are available, the Hazen method gives more dependable results and should be used.

In estimating the dependable supplies or safe yields for the Catskill and Croton watersheds, the mass diagram method was used since there were available runoff records covering 47 years for the Catskill sources and 86

years for the Croton. In planning the mass diagram for these watersheds, 25 per cent of reservoir storage was held in reserve and not added to the runoffs, leaving only 75 per cent of storage for maintaining flow. This was done for several purposes:

1. To provide a reserve storage for a drier period than any experienced in the past.

2. To allow for the effect of silting which has probably reduced the available reservoir capacity. (The extent of this reduction is not known.)

3. To compensate for that quantity of storage which is unavailable at full aqueduct delivery. (Approximately 20 billion gallons cannot be drawn from the reservoirs at the full capacity of the aqueducts because of hydraulic conditions.)

4. To have a reserve supply if the spring runoff is delayed during a period as dry as that upon which the dependable supply is based. (In this situation, although billions of gallons might be frozen on the watersheds, the reservoirs would be empty if no reserve storage were provided.)

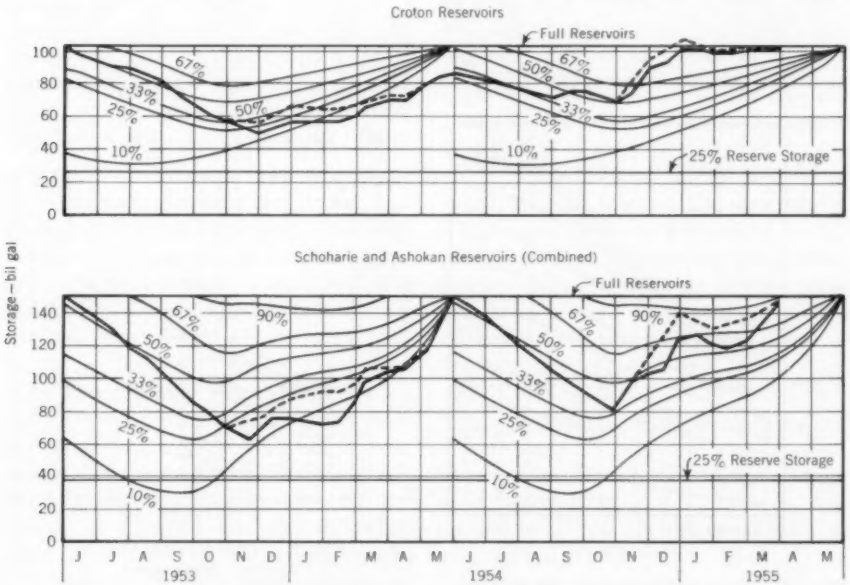

Fig. 4. Control Chart for Catskill and Croton Supplies

Actual storage in the reservoirs is indicated by the heavy lines. Lighter curves represent amount of storage required at any given date to insure full reservoirs on succeeding June 1, assuming various percentage years as indicated. Dotted lines for winter months (November–May) represent "potential runoff," corrected to include frozen storage on watersheds.

Safe Operation of Reservoirs

Since neither the advent nor the severity of a drought period can be predetermined, maximum security requires that the draft from a reservoir should never exceed the dependable supply

rate except when the reservoir is overflowing. Obviously, by this criterion, a large quantity of water is wasted over the reservoir spillway during normal and above-normal precipitation and runoff years.

A study of the mass diagram of runoff from a catchment area such as the Catskill or Croton shows that the storage necessary to sustain the dependable supply varies for different

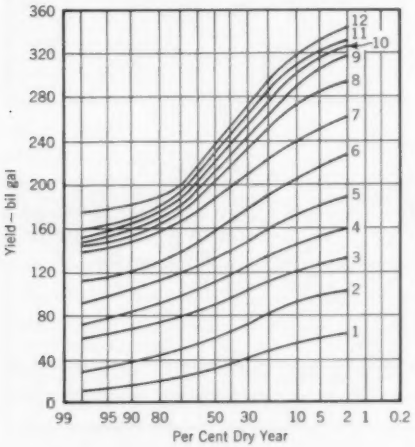

Fig. 5. Probability Curves of Runoffs, Catskill Watersheds

Curve numbers indicate periods of reference. Thus, 1 is curve for month of May alone, 2 for May and preceding April, 3 for May through preceding March, and so forth. Curve 12, therefore, represents 12-month period of May through preceding June.

months of the year. Thus, although the mass diagrams of the Catskill and Croton systems show that the reservoirs must be full at the beginning of a drought period in order to sustain a dependable supply throughout that period, they also show that, in every dry period, full reservoirs would have been required in the spring only, and not during other months of the year. This observation led to the development of a series of reservoir storage curves, called "control curves," to determine the most efficient utilization of the Catskill and Croton systems.

The control curves, shown in Fig. 4, show the storage necessary on any date in order to insure that the reservoirs will be full on the succeeding June 1, assuming various percentage years * and drafts equal to the maximum capacities of the aqueducts.

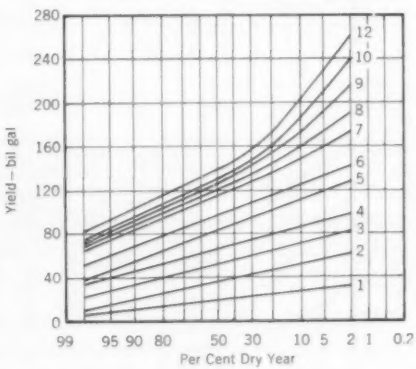

Fig. 6. Probability Curves, Croton Watershed

The periods indicated by numerals were derived as for Fig. 5. Based upon periodic runoffs for years of record, curves show periodic yields for various percentage years.

The curves were developed in the following manner:

1. Runoffs for the desired periods were obtained from available records. The periods used, starting with the month of May and going backwards, were: May; then the sum of May and

* The term "percentage year" refers to the ratio between the number of years a given yield has been equaled or surpassed and the total number of years of record. For example, a "99 per cent year" is one whose yield has been reached or exceeded in 99 out of 100 years.

April; then the sum of May, April and March; and so on, until the runoffs for the twelve months—May through the preceding June—were tabulated for each year of record.

2. The runoffs for each of the 7 months from November through May were adjusted as "potential runoff" to include the "frozen storage" on the watersheds, and were based upon the percentage of precipitation yielded for this period (November—May) each year for the years of record.

3. The periodic runoffs for the years of record were then arranged in order of magnitude, forming an array, and plotted on probability paper (*see* Fig. 5 and 6). A series of twelve probability curves resulted, from which the yields for the months or periods for various percentage years were obtained.

4. The monthly points on the control curves showing the amount of storage required in each percentage year considered, were determined by deducting the total drafts between the month in question and the following June 1 from the corresponding yields, and plotting the differences as depletions below full reservoirs on June 1. For example: From the probability curves of runoffs, the yield from the Croton watershed for the month of May, for a 50-per cent year, is 19.5 bil gal, and the total assumed draft is 375 mgd multiplied by 31 days or 11.5 billion gallons; therefore the amount of allowable depletion from a full reservoir is 19.5 minus 11.5, or 8 bil gal, giving the May 1 point on the control curve for a 50-per cent year. Similarly, the runoff for the months May and April for a 50-per cent year is 36 bil gal; the total draft is 375 mgd multiplied by 61 days (the number of days between April 1 and June 1) or 23 bil gal; therefore the amount of allowable depletion from a full reservoir for April 1 is 36 minus 23 or 13 bil gal. This procedure was repeated in determining the allowable depletions for all months for a 50 per cent year and a smooth curve was drawn through these points. Similar procedures were followed in plotting curves for the 90-, 67-, 33-, 25-, and 10-per cent years.

In the use of the control curves, the actual storage in the reservoirs is plotted on the charts, except for the 7-month period, November–May, for which the storage is adjusted to include the equivalent water content of snow, ice, and frost on the watersheds. The reason for using these dates is that the frost usually sets in about November 1, and all of the spring yield due to snow, ice, and frost usually runs off by the end of May. The difference between the adjusted and actual storage is the so-called "frozen storage." The percentages to be applied to the water equivalent of the precipitation in order to compute the frozen storage is obtained from charts, such as that shown in Fig. 7, based on studies of past records.

Frozen-Storage Curves

Beginning with the month of November, the percentage of precipitation yielded in that single month was correlated with that yielded for the 7-month period, November through May. For example, the percentage of precipitation yielded in November 1910, was plotted against that yielded for the 7-month period November 1910–May 1911. Similarly, the percentage of precipitation yielded in November 1911 was plotted against that yielded for the 7-month period November 1911–May 1912. This was done for the entire period of record, resulting in a chart with a wide scattering of points. In drawing a curve through

these points, considerable weight was allowed on the conservative side. The process of correlating the percentage of precipitation yielded was repeated for the sum of months, November and December; November, December, and January; and so on, until all 7 months, November through May, were similarly considered and curves drawn for each group of months.

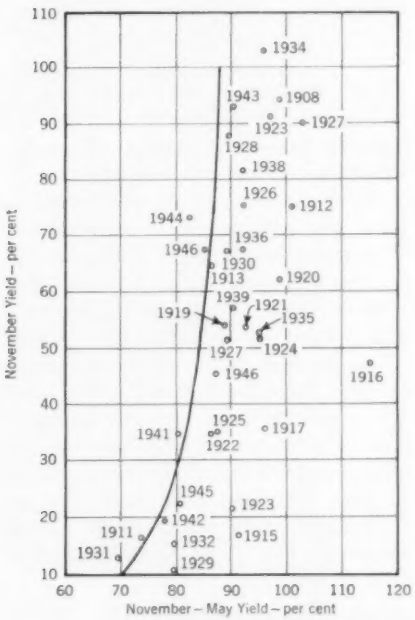

Fig. 7. Percentage of Precipitation Yielded in Winter Months, Esopus Watershed

Points for years of record represent percentage of precipitation yielded for single month of November plotted against yield for 7-month period, November–May. The curve, used to determine probable runoff for frozen-storage evaluation, was drawn conservatively.

The following example will serve to illustrate the use of the curves for determining the frozen storage. The precipitation on the Esopus watershed for November 1954 was 6.97 in. (equivalent to 31.1 bil gal over the watershed area). The actual yield was 17.9 bil gal or 57.5 per cent of the precipitation. The chart indicates that, for a runoff of 57.5 per cent in November, the probable runoff by the end of May will be 85 per cent of the precipitation, or 26.4 bil gal. Inasmuch as the actual yield was 17.9 bil gal, the frozen storage at the end of November was 28.4 minus 17.9 bil gal, or 8.5 bil gal.

In utilizing the control curves an attempt is made to have the storage in each of the watersheds fall on the same percentage year. This is done by increasing (if possible) or decreasing the draft from one of the watersheds in order to equalize storage with the other (with reference, that is, to the percentage year). By this method, the reservoirs in each of the watersheds have the same chance of filling by the succeeding June 1.

The control curves have been of great value as a guide in the operation of the New York City system, especially during the droughts of 1949–50 and 1953–54. By the use of the curves, a great deal of pumpage was curtailed, resulting in considerable financial savings.

High-Rate Recharge of Ground Water by Infiltration

Max Suter

A paper presented on Oct. 19, 1955, at the Iowa Section Meeting, Des Moines, Iowa, by Max Suter, Head, Eng. Research Subdiv., State Water Survey, Urbana, Ill.

THE need for study and solution of the problem of high-rate ground water recharge by infiltration has been encountered in Peoria, Ill., where approximately 60 mgd of ground water is pumped from glacial drift and alluvium above an impervious shale bedrock. As a result of this pumpage, ground water levels have been receding at such a rate that in the industrial area today they are about 20–25 ft below the pool level of the Illinois River. Wells located on territory with high bedrock are already dry or much reduced in their yield. From the recession measured over 10 years of close observation, it was estimated that pumpage in excess of the natural recharge was about 8–10 mgd. The Illinois River, which has a bottom that is highly silted, contributes nothing to the recharge at the regulated-pool stage and only little during floods.

Relief Measures

Several measures were available to reduce or compensate for the overpumpage, and even to build up a reserve for municipal and industrial growth. Possible measures for the reduction of ground water pumpage included replacement of ground water by river water, and recirculation of used ground water (including eventually conservation by means of cooling towers). Development of new well fields and new recharge techniques was also considered.

Although industrial pumpage was successfully reduced, part of the benefits thus achieved was offset by an increase in municipal pumpage. Nor did the development of new well fields prove a feasible relief measure. Although new well fields of high yield do exist in the neighboring area, they are either too remote or else yield water of unfavorable composition (high in iron and manganese). The water of the deeper bedrocks is likewise unsuitable because of high salt content and the presence of hydrogen sulfide.

Artificial recharge was, therefore, considered an important remedial measure. Only the water of the Illinois River is useful for this purpose. The side creeks are flashy and sometimes nearly dry, whereas the river has a minimum flow of about 5,000 cfs.

The quality of the Illinois River water has been thoroughly studied. Locally this river water still has, in general, a bad reputation, derived from conditions existing 30–40 years ago, when the river was badly polluted and void of game fish. For the last 20 years, however, the river has been appreciably cleaner. The dissolved-oxygen content never goes below 5.0

ppm and is often at saturation level. The bacterial count is normally about 26,000 per milliliter, and it was found that, above the town, about 3 per cent of the 1-ml sample tubes have no *Esch. coli*. The turbidity is less than 100 ppm for an average of 300 days per year. After consideration of these data and the cost of filtration, it was decided to try artificial recharge using chlorinated river water.

In studying the known methods of recharge, however, it was found that insufficient ground was available for filtration operations at the common rate of 0.5 mgd per acre. The need for considering possible means of achieving a higher rate of recharge was evident.

Some preliminary tests made in 1941 in an abandoned gravel pit on a local industrial property showed that, with a pumping rate of 1,400 gpm, infiltration rates of 37 mgd per acre could be obtained. This was in clean gravel, with less than 0.5 per cent passing a 200–mesh screen. As a result of these tests, high-rate infiltration was considered an important possibility, and efforts were subsequently made to determine the suitability of the raw water and the design of the pit required.

When, in 1947, funds became available for a hydraulic laboratory in Peoria, engineers found a lot bordering the Illinois River, on which a small recharge pit could be built. Through the Peoria Association of Commerce, local industries contributed about $75,000 for the construction of a larger pit which could be employed not only for research but also, if successful, for recharging a commercially useful volume of water. It should be noted that the state of Illinois, although concerned, of course, that its industries have sufficient water, plays no part in furnishing it, but is active only through its contributions to research in this field.

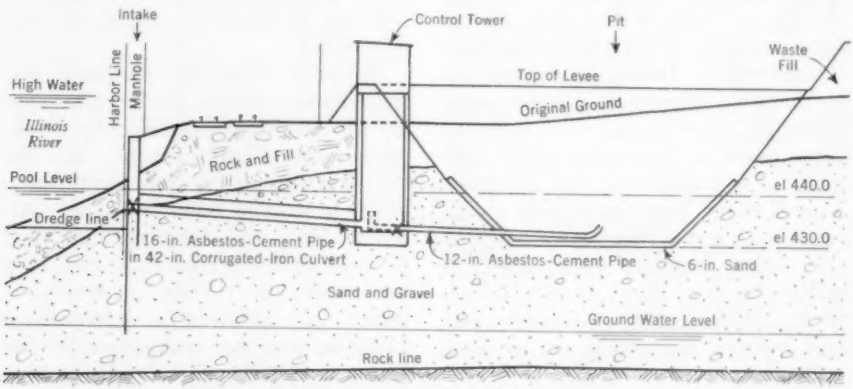

Fig. 1. Profile of Recharge Pit

In the fourth season of operation (1954–55) the sand filter material was replaced with 4–5-mm pea gravel, more than doubling total inflow per season.

Pit Design and Construction

It was hoped that a pit could be built to carry an average annual inflow

of 1 mgd. As the pit was expected to operate only during the 6–7 months when the Illinois River water has a temperature of less than 60°F, the daily inflow during the operating season would have to be about 2 mgd.

Test drilling at the proposed site showed that the material was much more silty than that of the 1941 gravel pit, with 2.1 per cent passing a 200-mesh screen. The design, therefore, provides only for an inflow of 15–20 mgd per acre.

The pit, shown in profile in Fig. 1, has a 40 × 62½-ft bottom (2,500 sq ft) at an elevation of 430 ft, or 10 ft below the pool stage of the Illinois River. This is still 10–12 ft above the ground water level and allows inflow by gravity from the river. The slopes up to el 442 are 1:2 and have, as does the bottom, a 6-in. layer of filter material. The higher parts are at a 1:1.5 slope.

The intake consists of a sheet piling at the harbor line, as shown in Fig. 2, with a coarse screen and a 16-in. pipeline (with valve) to the control tower. The top of this pipe at the intake is 4 ft below pool level to avoid ice effects. The control tower contains chlorinating and measuring equipment, fine screens, and a valve.

During the operating season hourly water level readings are made on the river, at the control tower, and in the pit; river water temperature is recorded; and a small sample is collected for a composite chemical analysis and turbidity test. Dissolved oxygen is measured daily, and bacteriological tests are made 4 days a week. Temperature, DO, and chemical composition are also determined daily from a sampling of a well in the laboratory building. In addition, ten observation wells around the pit (from 50 ft to ½ mile distant) are equipped with level recorders, and four nearby industrial wells are also used for water level measurements. All these activities provide a record of the operation of the pit sufficient for research purposes.

Filtration Techniques

The pit has been in operation for four seasons and is ready for a fifth. During the first 3 years 1-mm sand was used as filtering material. This sand clogged considerably because of the silt in the river water and, in the first year, had to be replaced twice. Although this procedure was costly, it produced an average inflow of 1.78 mgd or a rate of 12.5 mgd per acre. In more recent years fresh filter material was introduced only before operating the pit. Cleaning with a swimming pool cleaner was attempted; the goal, of course, was to remove the silt, but not the sand. This required many adjustments of the bearing surface of the cleaner and of the rate of pumpage. In spite of all this work average rates of inflow of only 1.03–1.05 mgd were obtained. This is the equivalent of 7 mgd per acre, about half the designed rate.

A radical change was made in the 1954–55 season. Instead of sand, 6 in. of 4–5 mm pea gravel was used as filter material. Some engineers warned that it was too coarse to filter out silt, and no references to its previous use for filtration were found. Tests of this material, however, showed that a considerable reduction in turbidity occurs when water passes slowly through it. For each 1-mgd inflow into the pit, the velocity through the filter is only 0.012 fpm, which is the necessary slow flow. The use of pea gravel as filter material was a success, raising the average inflow to 3.08 mgd, or about 22 mgd per acre. This is the average for the

whole season. The reduction of the normal inflow (excluding effects of floods) was only 12.5 per cent with the gravel, compared to 58 and 64 per cent experienced in seasons when sand was used.

Proof that the pea gravel removes the silt was also obtained by special tests made at the time of shutdown, when a thorough inspection was possible. It was found that the silt builds up a coat 1–2 in. thick on top of the pea gravel. Some silt penetrates the gravel but settles out mainly in the top layer; and no silt was found in the underlying original ground. As a quantitative test, 100 ml of the wet material from different layers at six different places in the pit was washed with distilled water. The wash passed through a 40-mesh screen, and the residue in the filtrate was weighed after evaporation.

The top silt contained 50.2 g of dry material, the top 2-in. layer of gravel 16.3 g, and the bottom 2-in. layer, 6.8 g. A saturated mixture, in which all pores in the gravel were filled with silt, held 20 g of dry material. These figures indicate that the top material still had open pores. Where the swimming pool cleaner had passed, the top 1 in. of gravel held only 2.4 g of dry silt.

Quality Control

In the first three seasons the chlorination dose averaged 8.8 ppm. This amount was used for safety, as it produced a water of satisfactory quality at the well. For economy, however, the least possible amount that would produce the desired result should be used. Applying chlorine at various rates showed that a water of satisfactory quality could be obtained with only 3 ppm chlorine. When only 2 ppm was used, the number of bacteria in the water at well No. 19 in the basement of the laboratory almost tripled. When no chlorine was used in a 3-day test period, the bacteria count increased considerably. A 10-ppm chlorine dose was used after this test to disinfect the soil and ground water.

Bacteria counts were also made on river water samples taken 4 days a week. The counts varied between 12,000 and 66,000 per milliliter with an average of 26,000. Some tests were made for *Esch. coli*. In previous years, when a high chlorine dosage was used, *Esch. coli* could often be confirmed with BGB media; during the last season, however, when a smaller chlorine dosage was used, no such confirmation was found. Also of interest is the fact that, following a period of rain and high turbidity in the river, the total bacteria count dropped after the first day or two, even though the turbidity remained high. Thus, high turbidity and high bacteria counts are not necessarily concurrent: rather, the first rain appears to flush a large number of bacteria into the river, but they soon die or are carried downstream.

Chemical tests demonstrate minor differences between river water and ground water, so that the effect of the recharge on the composition of the ground water cannot always be traced, especially such factors as hardness, alkalinity, sulfates, and chlorides. The river water contains dissolved oxygen, but the normal ground water does not. The first indication of an effect of the recharge is the disappearance of iron in the well water near the pit; later, some dissolved oxygen appears. After shutdown, the reverse process occurs.

Traces of residual chlorine were found occasionally during the period of operation, but these could not be correlated with the daily chlorine dosage. The highest residual chlorine

(up to 0.1 ppm) was obtained after shutdown of the pit.

Operating Procedures

It was originally planned to use the river water only when its turbidity was less than 100 ppm. During periods of higher turbidity, however, a shutdown not only causes an immediate loss of inflow, but also may prolong this loss by the freezing of ground in the emptied pit. On one occasion, a full month of operation was lost in this way. With the use of the cleaner, the high-turbidity clogging effect is reduced, and turbidity factors no longer require operating attention.

Special water level gages were developed for use in the pit. These read to 0.1 ft and have the 1-ft interval painted alternatingly red and white: thus, they can be read with certainty from long distances and during snow storms, even where set out in the water.

In the construction and preseasonal reconstruction of the pit, care must be taken not to compact the bottom. The construction must be done in a somewhat retreating way, so that no bulldozers or draglines need travel over the finished bottom; otherwise the soil may be compacted to a hard impervious material. In the first two years of operation, the Peoria pit bottom had to be loosened several times by blasting.

Clogging of the screens can occur easily. The fine well-screen is usually cleaned by a spray ring: however, when small dead fish lock in the slots of the screen, they cannot be removed unless the velocity through the slots is less than 0.6 fps. The best procedure then is to close the valve, remove the fish and resume operations: a task which can be done in 5 min.

From the hourly measurements taken, it was possible to plot the loss of head against the flow resulting from such shutdowns. The points show a wide spread, but also a limiting line. This line represents the minimum loss of head for a certain flow. Whenever a plotting falls on this line it indicates that all elements building up the loss of head—pipelines, bends, valves, screens, venturi tube, and so forth—are in perfect order. Unfortunately, few points fall on the line, and this is ordinarily an indication that the screens are partially clogged.

Comparison of Results

When comparing the operations during the various seasons, use can be made only of the average of the results in the second and third seasons, when sand was used as filter, and those of the fourth season, when pea gravel was used, as only these periods had uniform conditions.

Periods when the river level is above pool always cause an increase in the rate of recharge, although the reasons are not yet clear. For example, a 4-ft rise in the river nearly doubled the inflow, although causing only a 0.1-ft rise in the level in the pit. It is difficult to believe that this small rise in the pit level, which increased this wet area about 2 per cent, could cause an 82 per cent increase of the flow. Inasmuch as flood periods are completely irregular and beyond experimental control, comparisons of the various years have to be made carefully.

The average inflow in the second and third seasons was 1.04 mgd and the average total inflow per season 211 mil gal. In the fourth season an average inflow of 3.08 mgd and a total inflow of 508.4 mil gal were obtained. Of this increase, 87.5 mil gal was the

effect of having the river level above pool level; 38.1 mil gal, the result of cleaning the pit with the swimming pool equipment; and 219.9 mil gal, due to the use of pea gravel instead of sand. Thus it can be seen that the use of pea gravel had the greatest effect on increasing the rate of flow.

The use of pea gravel alone, however, does not produce a high rate of recharge. Much of the effect was also due to the type of pit, in which the wet side area is larger than the bottom area. Some simple reasoning shows that only as much water can flow through the bottom and walls of the pit as can flow away from the pit unless there is a special collecting tunnel underneath, as in Des Moines. The flow through the sidewalls is nearly unrestricted, whereas the flow through one small section of the bottom is always under some backwater effect caused by the flow in the neighboring section. Thus, the larger the bottom area, the smaller the inflow per unit area through the bottom.

Experiments on a model show that this flow per unit area reduces almost proportionally to the square root of the area, whereas the flow through the walls is proportional to the wall area itself. High rates of inflow can therefore be obtained by pits with small bottom and large side areas—that is, with deep water. Tests on the model of the Peoria pit show that, under its conditions, three-fourths of the inflow occurs through the side areas, although these represent only two-thirds of the wet area. Flow through the bottom is equal to that which can flow away around its circumference. In a large bottom, it is probable that more water can infiltrate than can flow away; and in a very small bottom, that more water can flow away than can infiltrate. The ratio depends in part on the inflow facilities and the permeability of the soil. The highest rate, however, occurs when both flows are used to their maximum capacities—although the theoretical conditions for such a design have not yet been established. On the other hand, the flow through the sidewalls has a much smoother flow-away path and it is here that the entrance conditions govern the rate of inflow.

It is believed that a great part of the success obtained with pea gravel is due to its facilitation of the passage of water into the ground. How this is done is not yet clear, but it may be that it provides a suitable transition flow between the open water and the finely distributed ground water flow, and thus reduces entrance resistance.

Much more could be said, if space allowed—about the effect of ice, of algae, of temperature phenomena, and of the characteristics during emptying of the pit, as well as of other phases, all connected with the problem of high-rate recharge, either by reducing it or by verifying its effectiveness.

A recharge pit with the high rate of 20–25 mgd per acre was achieved in Peoria. Much has been learned of the methods by which this was accomplished, but much also cannot yet be explained. It is hoped that future investigations may help to further understanding.

The Lawn-sprinkling Load

Panel Discussion

A panel discussion presented on Jun. 16, 1955, at the Annual Conference, Chicago, Ill.

Long Island, N.Y., and Levittown, Pa.—Angus D. Henderson

A paper presented by Angus D. Henderson, Cons. Engr., Westbury, N.Y.

CHANGING times and the long-continued period of prosperity and better living have intensified the old problem of lawn sprinkling. In the past, lawn-sprinkling requirements occasionally overtaxed a municipal supply system. In suburban areas, it has recently become necessary to design the entire system in accordance with the requirements for lawn sprinkling. These conditions are in contrast with the practice of 20 years ago, when the controlling factor in the design of a system was the quantity of water that might be required to meet the needs of a bad fire. Today, when a system is designed to meet the lawn-sprinkling requirements, a sufficient capacity is almost automatically insured for any fire load that may be imposed.

Long Island

Over the past few years, data have been collected that show the relationships between the average annual demand, the average demand for the maximum month, maximum week, maximum day, and maximum hour for a number of systems on Long Island, N.Y. Some of these representative relationships are shown in Table I.

The Manhasset-Lakeville Water District, serving an area of 10 sq miles and a population of about 30,000 people in northwestern Long Island, had an average consumption in 1953 of 5 mgd. Consumption for the maximum month of that year was 8.1 mgd; for the maximum week, 10.1 mgd; and for the maximum day, 11.8 mil gal. The rate for the maximum 6-hr period was 20 mgd; for the maximum 3-hr period, 23 mgd; and for the maximum hour, 25 mgd. This maximum hourly rate is approximately five times the annual demand.

In this particular district over the past five years, meeting the lawn-sprinkling demands has necessitated a capital investment of more than $1,000,000 for additional wells, additional storage facilities, and large transmission mains. An end to these expenses is not yet in sight because the demand for lawn-sprinkling water is continuously and rapidly increasing.

The Citizens' Water Supply Company of Great Neck has generally been unable to meet its peak loads because of a special lawn-sprinkling problem, which, at the moment, is confined to its local area but which will undoubtedly spread to other parts of Long Island and to suburban areas adjoining all large cities in the Middle Atlantic states.

Whereas the Manhasset situation has been created primarily by hand hoses and portable hose-connected sprinklers, the problem in Great Neck has been aggravated by the installation of underground lawn-sprinkling systems. Some 200 such systems have been installed with permanent piping and sprinkler heads underneath the lawns, many of which create demand rates of more than 70 gpm each. The Great Neck water company has been required to invest more than $300,000 to service these particular sprinkler installations, or about $1,500 per installation.

Regulating the use of these systems has been rather ineffective, even though time clocks have been installed and sprinkling by the underground systems is not permitted except during the night hours. The result, however, has been that the storage facilities have been drawn upon heavily during the night and the storage tanks have not had a chance to refill. The daytime draft is then taken from the already partially empty tanks.

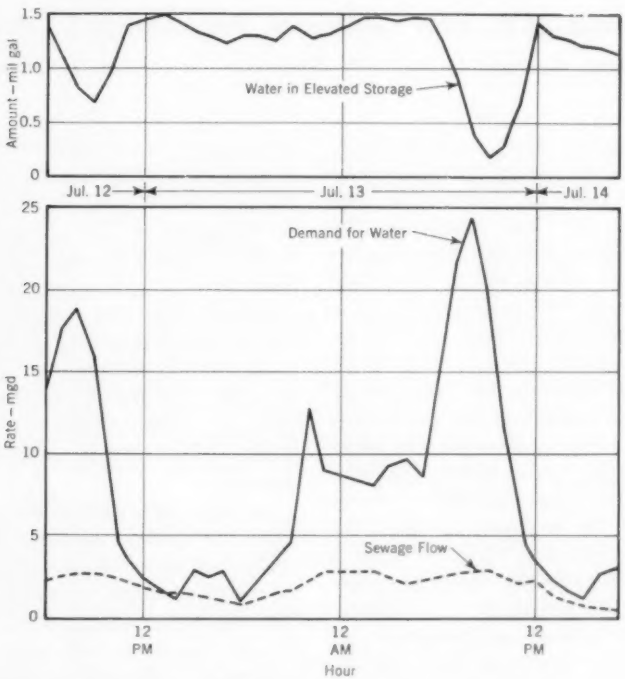

Fig. 1. Demand, Storage, and Sewage Flow Data for Maximum Day at Levittown, Pa.

The peak day at Levittown in 1954 was Jul. 13. The hourly demand rates, changes in storage, and the sewage flow for this day are shown.

Levittown

The construction of a large housing development at Levittown, Pa., provided the opportunity to secure very detailed data about lawn-sprinkling demands in a new and almost entirely residential community. The ultimate development of Levittown will create a community of approximately 17,500 homes with a population of about 70,000.

The water supply system was designed before construction started, and a daily demand, averaged over the year, of 100 gal per capita was assumed. For design purposes, it was also assumed that the maximum day would be approximately twice the average day and that the maximum hourly rate would be four times the rate on an average day.

In June of 1954, when the actual population in Levittown was approximately 30,000, warning signals began to appear and, for a period of some 3 months, an hourly record of water pumped, change in storage contents, total demand for water, and sewage flow was maintained. As a result of these records, it was found that the maximum hourly rate was approximately ten times the average daily demand for the year, and it became necessary to install additional storage facilities which were placed in operation this summer.

The total hourly demand rates, the change in elevated storage, and the variations in sewage flow for the maximum day of 1954 are shown in Fig. 1. The difference between the sewage flow and the total demand provides a reasonable approximation of the quantity of water which was used for watering lawns.

It is true, of course, that water supply systems are in the business of selling water and that, if their consumers are willing to pay the costs, they should reasonably expect to be able to secure from the water supplier all the water which they desire to purchase. The question that arises is how the capital investment required to meet the lawn-sprinkling demands should most properly be financed.

An analysis of 1,000 hr of peak use at Levittown during the summer of 1954 provided some rather startling cost and revenue data. The average consumption for the year was approximately 2.4 mgd and, without installing

TABLE 1

Ratio of Peak Demands to Average Annual Demand

Item	Manhasset-Lakeville	Port Washington	Westbury	Citizens Water Co., Great Neck	Albertson	Levittown, Pa.
Avg. for year	1.0	1.0	1.0	1.0	1.0	1.0
Max. month	1.63	1.76	1.55	1.77	2.5	2.0
Max. week	2.05			2.24		
Max. day	2.35	3.0	2.5	3.0	3.75	3.0
Max. 6 hour	4.0					
Max. 3 hour	4.6	3.4	3.5	4.9		8.75
Max. hour	5.0	4.1	4.5	5.65	8+	10.0
Restricted use	No	Yes	No	Yes	No	No

additional storage, the system as designed could easily meet a peak of four times the average consumption.

The quantity pumped at times when the rate was in excess of 10 mgd totaled only 63 mil gal in the entire year. If it is assumed that the water was sold at a price which would provide $100 per million gallons to meet fixed charges after the payment of operating expenses, the money available from the sale of water on peak loads which could be used to defray the expense of the additional facilities required to meet such peak loads would be about $6,300 per year. In this particular situation, the most economical way of meeting the peaks is by the provision of additional storage, and the fixed charges on this investment will be somewhere between $30,000 and $40,000 per year.

One solution to the financing problem may lie in a general increase in rates, and in spreading the cost of meeting lawn-sprinkling demands over all the users of the community. Such a procedure, however, does not seem fair and equitable to those consumers such as stores, light industries, and apartment houses, who do not use water for lawn-sprinkling purposes.

At least one private water company is on record as discouraging the use of meters on residential premises because of the lawn-sprinkling problem. This company believes that a frontage and fixture charge provides better sources of revenue inasmuch as it has many consumers who pay an annual fixed charge for each hose bib.

It might be suggested that the best means of securing the required revenue to pay for the capital improvements necessary to meet lawn-sprinkling loads possibly consists in metering all residential premises and also superimposing a moderate surcharge for hose bibs and a much heavier surcharge on undergound lawn irrigation systems. If this were done, the charge for an underground sprinkler system should, of course, bear a direct relationship to the demand which such a system might create.

Indianapolis, Ind.—*L. S. Finch*

A paper presented by L. S. Finch, Vice Pres. and Chief Engr., Indianapolis Water Co., Indianapolis, Ind.

The magnitude of the lawn-sprinkling load upon a municipal water supply system not only varies from season to season, month to month, day to day, and hour to hour, but it also changes from city to city, and even from district to district within the same city. Consequently, an attempt to do more than merely generalize as to lawn-sprinkling characteristics and magnitude is futile.

Because the author is most familiar with lawn-sprinkling loads at Indianapolis, the data included in this discussion will be based upon Indianapolis experience, although the comments and conclusions which are presented are intended to be of general application.

Sprinkling-Load Factors

At Indianapolis the sprinkling season may be expected to extend from the middle of May to the middle of September, or even later in a dry year. In a year of better than average rainfall, the sprinkling load may be negligible until well into June, although very heavy loads may usually be antici-

pated in that month because that is when homeowners sprinkle freely to establish their lawns and gardens. With continued drought or extremely hot weather, however, July and August loads may often exceed those of June.

With the advent of daylight-saving time, the length of the daily peak sprinkling periods has increased. "Daylight-saving time" could, in fact, be called "sprinkling-extending time," for the extra hour of daylight means an extra hour of sprinkling.. This extra hour results in greater depletion of distribution storage. As the days become shorter during late July and August, even though the peak loads may be as high, storage is conserved because the load drops off earlier in the evening.

The most significant variations in sprinkling-load characteristics, other than those relating to the season and the weather, are those resulting from different cultural development. Sprinkling-load characteristics, for example, are quite different in a highly industrialized city or district than they are in a predominantly residential district.

At Indianapolis, the water works is divided into three separate primary-pressure districts, the distribution system of each district being completely isolated from the others by valves. That these three districts vary widely as to occupancy and use characteristics may be seen from the following descriptions.

Central District

The Central District is predominantly industrial and commercial. It includes the main business district and many commercial and industrial installations. Large air-conditioning installations practice water conservation either through the use of cooling towers or by using water from private wells rather than from the public system. Many moderate-size installations, however, use nonconserved water from the public system. Residential areas generally are congested and are made up of middle- and low-income bracket customers. In the Central District, water is pumped directly to the system and there is no distribution storage.

East District

The East District is a mixed industrial, commercial, and residential development and it includes several rather large industries. City water is more widely used for air conditioning than it is in the Central District, although the larger installations practice water conservation. Residential areas are made up predominantly of middle-income bracket customers living upon relatively small lots. In this district, a 1.5-mil gal elevated tank floats on the distribution system.

North District

The North District is predominantly residential, there being practically no industrial development in the area. There are numerous neighborhood commercial developments, however. Large air-conditioning units depend upon private wells, or recirculate the water over cooling towers if city water is used. Smaller commercial and residential units generally use nonconserved city water. Residential customers are in the middle- and high-income brackets and occupy larger lots than in either of the other two districts. The North District contains several golf clubs which depend upon city water for sprinkling, but the sprinkling is usually done after peak evening hours. In this district, as in the East

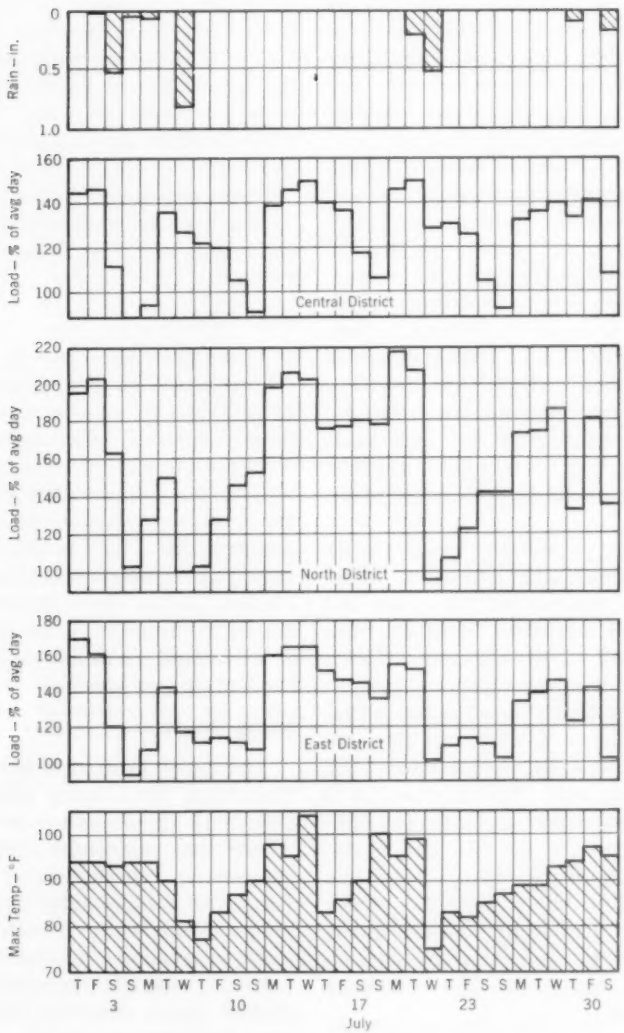

Fig. 2. Consumptions for July, 1954, Compared to Annual Average Daily Load

Daily water usage for the three districts, as percentage of load for average day in that district, is shown in relation to rainfall and maximum temperature. Average daily load for Central District for 1954 was 39.87 mil gal; for East District, 15.37 mil gal; and for North District, 6.90 mil gal

District, a 1.5-mil gal elevated tank floats on the distribution system.

Load Characteristics

As might be expected, lawn-sprinkling loads vary widely in these districts. In the Central District, after 4 or 5 PM and unless there is a lawn-sprinkling load to maintain the usage rates, consumption diminishes radically because industries and major commercial establishments are closing for the day. When the weather is hot or extremely dry, the lawn-sprinkling load is of a magnitude approximately to replace the industrial and commercial usage which has been discontinued.

During 1954, the all-time, peak, daily-consumption rates occurred in July. They resulted from relatively high temperatures and from several days without rain. On July 14, the day on which the all-time system record was established, the temperature reached 104°. This resulted in the use of 99.27 mil gal of water. Consumption records also were established in the Central and East Districts. In the North District, while consumption was high on this day, it was exceeded 5 days later when the continued drought resulted in extreme lawn-sprinkling usage.

A statistical comparison of the load characteristics in the three districts is presented in Table 2. The data included in this table are sufficient for statistical purposes, but for design purposes or for analyzing the adequacy of plant and distribution system capacity, more detailed information regarding atmospheric temperature, rainfall distribution, and holiday and week end water use characteristics must be considered. It is the author's experience that such factors can best be related and evaluated from graphical representations. Also, if comparisons of peak rates are to be made between the districts within a city, or between experiences at different cities, a com-

TABLE 2

Consumption Rates—Jul. 14, 1954

District	Jul. 14 Use *mgd*		1954 Avg. Day Use—*mgd*	Jul. 14 Use/1954 Avg. Day Use %	
	Day	Max. hour*		Day	Max. hour
Entire city	99.27	150.99	62.14	159.8	243.0
Central	59.87	81.70	39.87	150.2	204.9
East	25.47	45.95	15.37	165.7	299.0
North	13.93	27.64	6.90	201.9	400.1

* Maximum hour was 6–7 PM in all cases except for East District, where it was 2–3 PM.

In this district, therefore, there is no pronounced peak period in the evening when lawn-sprinkling usage is at its greatest.

The North District is the opposite of this. Lawns in this district are large and there is no industrial load to be eliminated in the late afternoon. The evening sprinkling peaks in North District are extremely high, therefore.

In the East District, because the area approximates a composite of the other two districts, evening sprinkling peaks are more pronounced than in the Central District but are not as extreme as in the North District.

parison with the annual average daily rate can be more intelligently interpreted than can a comparison based upon million gallons of daily consumption. In this article, therefore, peak sprinkling-load analyses and comparisons will be based upon daily usage rates or hourly usage rates versus annual average daily consumption (*see* Fig. 2–5).

Accordingly, in Fig. 2, the July consumption in percentage of the annual average day, the maximum daily temperature experienced, and the inches of rain which fell have been plotted for the Central, East, and North Districts. Similar graphs for June and August were prepared, but only July was selected for further analysis since the peak sprinkling loads were experienced during that month.

An interesting comparison can be made between the usage and sprinkling habits of the East and North Districts and those of the Central District by noting the days of the week on which the peaks and valleys occurred. The Saturday and Sunday characteristics in the Central District, where there are relatively light sprinkling loads, may be noted merely by scanning the graph and selecting the valleys. This is also true in the East District, where there is considerable industry, but the valleys are not as pronounced because there is relatively more sprinkling. In the North District, however, sprinkling loads are seen to continue at a high

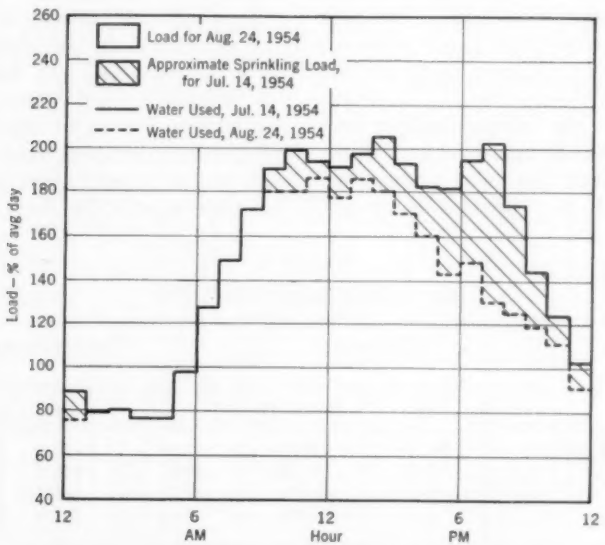

Fig. 3. Hourly Usage for Central District—Jul. 14, 1954

Load for the annual average day for the Central District was 39.87 mil gal. The maximum usage occurred in the afternoon in this district, when industrial and air-conditioning demands were greatest.

level even on Sunday. In fact, the only Sunday valley which appears on the North District graph occurred on Jul. 4, a major national holiday.

By way of contrasting Saturday and Sunday usage in the districts under different weather conditions, compare the conditions for the period between Thursday, Jul. 15, and Sunday, Jul. 18. In the North District, usage on Saturday and Sunday continued at the same level as on Thursday and Friday, whereas, in the Central District, usage on Saturday decreased markedly from that of Thursday and Friday, and fell still further on Sunday.

From these comparisons it might be generalized that, in a district predominantly industrial and commercial, minimum usage may be expected upon Saturday and Sunday in spite of sprinkling loads, whereas in a middle- and high-income district having no industry, Saturday and Sunday make little or no difference upon the load during a period of high sprinkling rates.

In comparing the peak loads in the three Indianapolis pressure districts, it may be seen that on Jul. 14, the date upon which the system-wide maximum day's usage was experienced, maximum daily usage in the Central District and the East District was 150 per cent and 165 per cent, respectively, of the annual average. In the North District, however, the rate was 200 per cent of average. Maximum temperature on this day was 104°, and it was

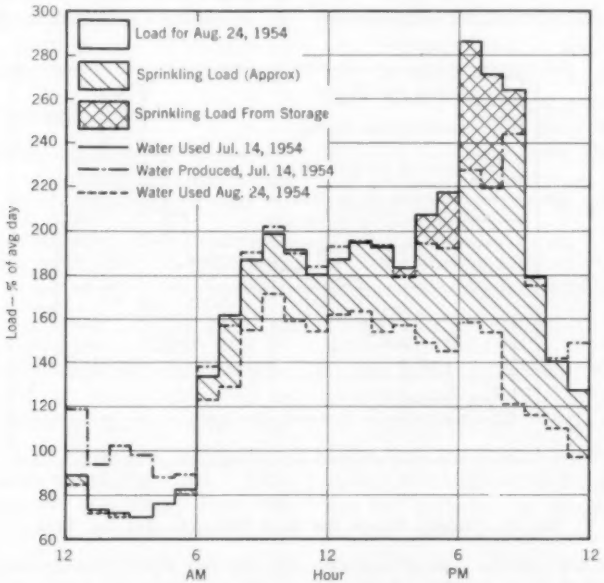

Fig. 4. Hourly Usage for North District—Jul. 14, 1954

Load for the annual average day for the North District was 6.90 mil gal. In this district there is no industrial load to be eliminated and evening sprinkling peaks are very high.

the only day in the month when the temperature exceeded 100°.

The North District peak, it may be noted, did not occur on the same day that the all-district peak was experienced, the maximum North District usage occurring on Jul. 19 with 219 per cent of the average. This can be explained by reference to Fig. 2 where it may be seen that both Jul. 14—when the peaks occurred in the Central and East Districts—and Jul. 19—when the peak was experienced in the North District—were hot, with maximum temperatures of 104° and 95°, respectively. On Jul. 14, however, only 7 days had elapsed since 0.75 in. of rain had fallen, whereas, on Jul. 19, 12 days had elapsed. This prolongation of the drought had little effect upon peak rates in the Central and East Districts because sprinkling is not their predominant use factor. Because lawn sprinkling is the controlling peak factor in the North District, however, the prolonged drought produced record usage rates in that district on the 19th rather than on the 14th.

While this discussion of the all-time peak day of 1954 is interesting, it does not sufficiently analyze the adequacy of plant and distribution capacity. It is of prime importance, however, to note from the analysis that 12 days of drought in 1954 produced a record peak demand in the North District largely because of the lawn-sprinkling and air-conditioning usage. It is interesting to note, too, that although the 12 days of drought in 1954 resulted in the establishment of record peaks, there were 35 days of dry hot weather in the summer of 1936. From this it may be deduced that, since

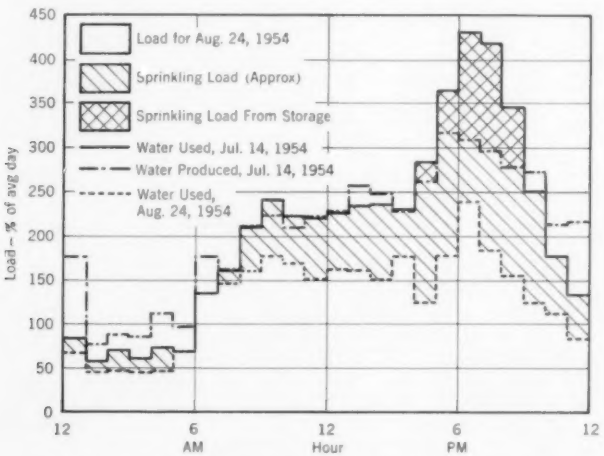

Fig. 5. Hourly Usage for East District—Jul. 14, 1954

Load for the annual average day for the East District was 15.37 mil gal. This district, having load characteristics which are a composite of those in the other two districts, has sprinkling peaks which are more pronounced than in the Central District but not as extreme as in the North District.

longer dry spells may be expected to occur, a much higher North District peak can be anticipated than that of 1954. The same also is true in a lesser degree with respect to the East District, where sprinkling and air-conditioning loads also are important factors. The men responsible for maintaining adequate water service in Indianapolis, therefore, have not become complacent merely because existing facilities were capable of meeting the 1954 all-time peak demands.

Air Conditioning

It is difficult to separate air-conditioning from sprinkling load, but it can be done with a reasonable degree of accuracy by comparing peak periods on hot, dry days with those of hot, rainy days.

From Fig. 2, the relation of peak pumping periods to weather can be seen. But no definite statement can be made from these charts alone as to what proportion of the excessive load resulted from air conditioning and what from the lawn-sprinkling load. That can be deduced, however, from an analysis of the daily-usage rates. To aid in this analysis, hourly usage on Jul. 14 in the three districts has been plotted in Fig. 3–5.

It may be seen that the daily usage characteristics of the three districts do not even remotely resemble each other. In the Central District, Fig. 3, the maximum hour occurred not in the evening, when sprinkling was at its peak, but in the middle of the afternoon, when industrial and air-conditioning demands were at their maximum. A high demand occurred in the evening hours but it did not exceed the afternoon peak. In the East District, Fig. 5, the peak hour occurred between 6 and 7 PM, when the after-dinner load coincided with the sprinkling load. Only slightly less was the sprinkling load between 7 and 8 PM. After that, sprinkling diminished until, between 9 and 10 PM, the load had decreased to that of the mid-day hours. In Fig. 4 the same characteristics may be noted for the North District.

As would be expected, the magnitude of the peak hourly rates, as compared with the average day's pumpage, varied widely between the districts. It may be noted from Fig. 3–5 that the peak hourly rates, as compared with the average annual daily rates, varied between approximately 200 per cent, 290 per cent, and 430 per cent for the Central, East, and North Districts, respectively.

While it was known from observations made at the time that the peak evening rates could largely be attributed to sprinkling, the comparative magnitude of the loads chargeable to sprinkling, as distinct from air conditioning, was not readily apparent. By plotting the daily load curve for a hot, rainy, day against the load curve for a day which was hot and dry, an approximation of the air-conditioning load, as distinct from the sprinkling load, was developed. A study of Fig. 2 revealed no such July day which was not affected by other abnormal circumstances, such as the Jul. 4 holiday. June experience was reviewed without success. It was finally determined that on Aug. 24 it had rained sufficiently to curtail sprinkling but not enough to cool the air to the point that air conditioners would be taken out of service or their use materially curtailed. Consequently, this day was selected for comparison with Jul. 14 in order to separate, as far as possible, the air-conditioning from the lawn-sprinkling load. It would have been desirable to plot sev-

eral such hot days when there was no sprinkling so that an average condition could be observed, but since only one day during the entire season had the proper characteristics, it was necessary to estimate the 1954 sprinkling load from that one day's experience. Even so, the sprinkling loads were somewhat less than indicated upon the graphs because some air-conditioning load is included with the sprinkling load. This results from the fact that more conditioning water was probably used on Jul. 14, when the maximum temperature was 104°, than on Aug. 24, when the temperature was 93°.

Sprinkling Loads and Distribution Storage

It may be noted on Fig. 4 and 5 that the portion of the consumption during the evening sprinkling-peak loads, which was supplied by the 1.5 mil gal tanks in the East and North Districts, has been indicated. This was done so that the probable effect of adding a second tank in each district on peak pumpage rates could be noted. It readily can be seen from these charts that, in each district, the addition of a second tank of similar capacity would reduce evening pumpage peaks to levels approximating those prevailing during the day. It goes without saying that this would result in operating economies and make it unnecessary to increase central-station pumping and distribution system grid capacity to meet the increasing loads, at least until they become much greater than at present.

Storage and Controlled Sprinkling

Many large cities have found it necessary to reduce peak sprinkling loads by requiring customers on opposite sides of the streets to sprinkle on alternate days. While this has not been necessary, nor is it contemplated at Indianapolis, it appears from Fig. 4 and 5 that the peak loads in the East and North Districts would be reduced by about the same amount by alternate sprinkling regulations (approaching 50 per cent reduction) as they would be by the addition of a second storage tank in each district. From Fig. 3 it can be seen that such regulation of sprinkling habits would not accomplish the same reduction of peak loads in the Central District as in the other two districts. This comparison has been drawn for illustrative purposes only and should not be taken to indicate that curtailment or regulation of sprinkling is contemplated at Indianapolis.

Conclusions

From city to city, conditions vary so greatly with the seasons, the weather, and the cultural development that to attempt to do other than generalize as to lawn-sprinkling load characteristics and magnitude would be futile. Very broad conclusions, however, may be drawn from a study of data which have been presented in this discussion, and even they must be used with care. The conclusions may be briefly stated as follows:

1. More pronounced lawn-sprinkling peak loads may be expected in cities and towns having predominantly residential rather than industrial characteristics.

2. The adoption of daylight-saving time results in lengthening of the evening sprinkling-load periods and greater depletion of distribution storage.

3. In industrial cities or districts the probable sprinkling load will not be

great enough on Saturday and Sunday to offset the week day industrial load, with the result that overall water consumption on these days will be the lightest of the week.

4. In residential areas, or cities with light industrial development, week end sprinkling loads during hot or dry weather will probably be sufficient to maintain the overall water consumption at a level equal to or higher than that experienced on week days.

5. In industrial cities, the peak hourly consumption loads may occur in the afternoon rather than in the evening. In areas more predominantly residential, peaks will normally occur in the evening during hot or dry weather.

6. In industrial cities, peak hourly water consumption rates may be as low as 200 per cent of the annual average daily rate, as compared to 400 per cent or greater hourly rates in areas predominantly residential.

7. Where pronounced daily peak periods are caused by lawn-sprinkling loads, the installation of distribution storage or adoption of alternate-day sprinkling programs generally can be expected to result in pumping economy and the possible deferment of pumping or distribution storage extension programs.

Kansas City, Mo.—*Melvin P. Hatcher*

A paper presented by Melvin P. Hatcher, Director, Water Dept., Kansas City, Mo.

To show the character of the lawn-sprinkling load in Kansas City, Mo., data relating to the maximum day in 1954 will be used. Load information for some other main classes of service will be included, notably the use of water for air-conditioning service.

Background

Kansas City directly serves 100,000 consumers, and it sells water on a wholesale basis to agencies that serve an additional 50,000 customers living outside the city. As in most metropolitan areas, the maximum daily demand for water has been growing at an accelerated rate in recent years. The record for the last 5 years is shown in Table 3.

Because the studies that have been made of maximum daily demand included a determination of the demands for air-conditioning service, some facts about the demands for this service were discovered. The use of water-cooled air-conditioning equipment in Kansas City has increased at a rapid rate in recent years. The total installed capacity in 1946 was about 22,500 tons. By 1954 the total tonnage had increased to 55,000 tons—about 117 tons per 1,000 population. In a recent engineering report it was predicted that total installed capacity would reach 383,268 tons by 1970.

Air Conditioning

Water demands of air-conditioning service, because there is a high ratio of maximum to average daily demand, have an unfavorable yearly load factor. With the rapid increase in the use of water for this service it became increasingly important that the service pay its own way, but this was not possible under the rates that were in effect in Kansas City prior to 1951. These and subsequent changes in rates have

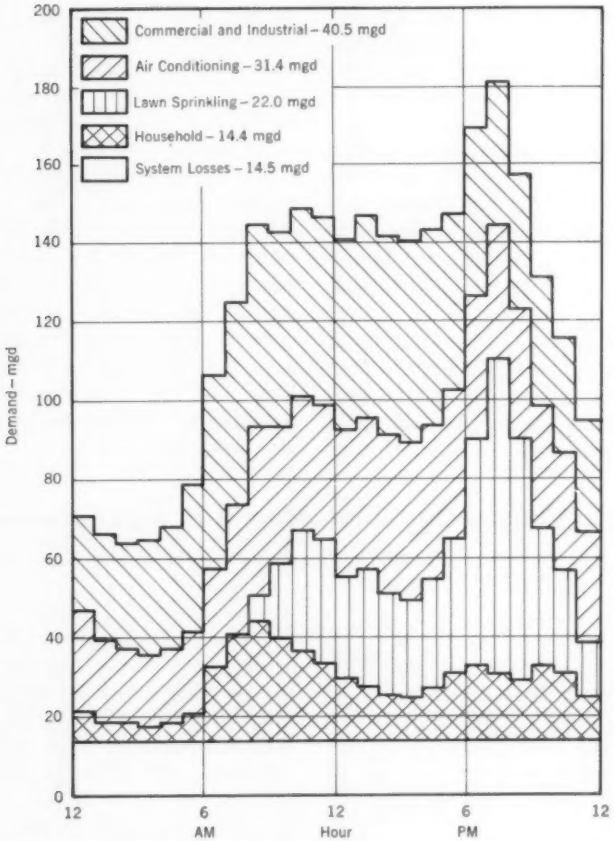

Fig. 6. Breakdown of Total Consumption for Jul. 12, 1954

The total consumption for the day was 122.8 mgd. Suburban and flowline consumption were not included.

dealt specifically with this service, but they have not yet managed to provide full compensation for its cost. Kansas City now has, however, an ordinance which prohibits the installation of air-conditioning equipment that does not provide for recirculation of the condenser-cooling water.

Lawn Sprinkling

The rate changes made in 1951 did not overlook the unfavorable–load factor effect of lawn-sprinkling service. Studies made in Kansas City indicated that this effect is quite like that found for air-conditioning service, the ratio of maximum to average daily demand being 7 or 8 to 1 as compared to the 1.25 to 1 ratio for regular household or commercial service. The present rate plan takes lawn-sprinkling service into account by means of a separate rate for single and two-family residences. This is a uniform rate which

includes no steps, and was formed on the basis that all water used over and above the small amount needed for household purposes is for lawn sprinkling or air conditioning and each of these special classes of service must bear a rate which reflects their unfavorable load factor.

The results of the study of maximum daily demand in 1954 are shown in Fig. 6 and 7. Figure 6 shows a composite of the total demand in terms of system leakage, household consumption, and the three classes of service shown in Fig. 7. The determinations begin with an hourly load curve developed from pumping and storage records to show the rate of use by customers inside Kansas City. This is a reasonably reliable curve in that it reflects the subtraction of a known suburban demand, which is determined from hourly records of flow, from a known total-system demand.

TABLE 3

Maximum Daily Demand in Kansas City, Mo., 1950–1954

Year	Max Daily Demand *mgd*
1950	98
1951	106
1952	122
1953	136
1954	154

The sprinkling-demand curve was developed from a difference between the demands on Jul. 12, 1954, the day of maximum demand, and the demands for two days later. Because of operating difficulties at one of the main pumping stations just after the day of maximum demand, it became necessary to impose restrictions on the use of water for lawn sprinkling. A considerable number of inspectors toured the residential areas to insure compliance and all violations were promptly stopped. The restrictions were therefore regarded as quite effective and the conditions bearing on the use of water, particularly temperature and lack of moisture, were practically identical with those of the maximum day.

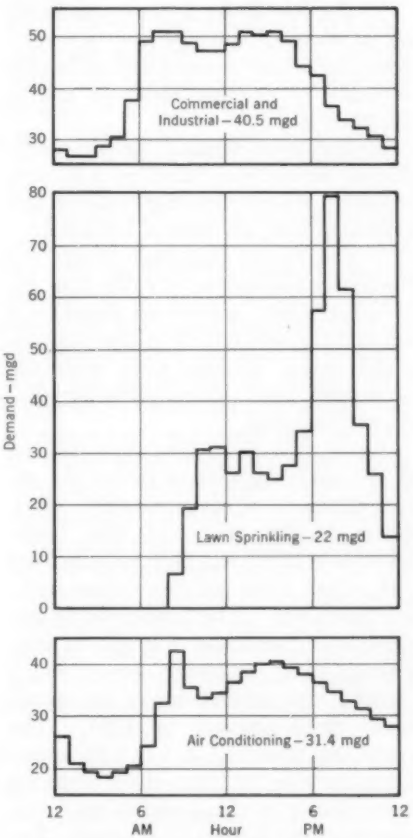

Fig. 7. Consumption by Three Classes of Service—Jul. 12, 1954

The data shown are for the maximum day of 1954.

The further evolution of the total demand curves on the maximum day and the hourly variation of these demands were as follows:

1. The household demand was based on a study of average use per residence for winter months and an hourly variation of demand based on use in apartments.

2. The total demand for air-conditioning service was based on a known, installed, air-conditioning capacity and estimated uses per unit of capacity.

3. The hourly variation curves for commercial-industrial service and for air-conditioning service are based on a cut-and-try subdivision of the total demand inside Kansas City with the household and lawn-sprinkling demands excluded.

Again, the lawn-sprinkling section of the study appears to reflect the actual demands rather accurately. Evidence of the shape of this lawn-sprinkling curve has been recorded for only a few cities but these show striking similarity. The Louisville Water Company distributed a graph in supplement to its last annual report which showed the subdivision of its maximum daily demand in 1954. The curve, as determined for lawn-sprinkling service for that city, conforms quite closely in shape and in total volume with the Kansas City curve. The total demand curve for Milwaukee, as reported by Edward F. Tanghe (*1*), indicates a similar relationship of the lawn-sprinkling load to the total load.

Because of its unfavorable load factor, the lawn-sprinkling demand is important in reference to rate making. The need for adequate revenue from this service is becoming increasingly important because of the growth in demand for this class of service. This increase has resulted from the outward growth of the metropolitan areas, greater average size of lawns, more flowers, more gardens, and a greater general prosperity.

The shape of the lawn-sprinkling curve—that is, the concentration of the demand in the evening hours—contributes to the difficulty of equalization. Milwaukee, for instance, is attempting to solve this part of the problem with scattered ground storage and booster pumps (*1*). All of these factors add to the growing mass of evidence that this is a very expensive class of service.

Reference

1. TANGHE, EDWARD F. Meeting Peak Demands at Milwaukee. *Jour. AWWA*, 47:60 (Jan. 1955).

Milwaukee, Wis.—*Edward F. Tanghe*

A paper presented by Edward F. Tanghe, Supt., Milwaukee Water Works, Milwaukee, Wis.

There are three present factors which will tend to increase the future lawn-sprinkling loads. These factors are as follows:

1. Consumers are moving from the older, more congested areas to the outskirts of cities and to the suburbs, which means longer distances for the water to travel from the pumping stations to the consumers.

2. The lots in the newer areas are invariably larger, with more expansive lawns.

3. The new areas are generally higher in elevation, which increases the difficulty of maintaining good pressures during peak demands.

Most lawn sprinkling in and around Milwaukee, Wis., is done between the hours of 5 PM and 8 PM. Following a

number of dry days, this lawn sprinkling creates a demand which has sometimes equalled half the capacity of the Milwaukee plant, although this high peak rate may not occur more than two or three times a year. Because the amount of water used for lawn sprinkling comprises only about 2 per cent of the annual pumpage, lawn sprinkling represents a very uneconomical use of the water supply.

Storage and Sprinkling Control

Milwaukee decided that holding half of the plant capacity available for the infrequent sprinkling use was entirely unwarranted. As a more reasonable solution, large storage units have been and are being installed quite some distance from the pumping stations. These units are filled overnight by the normal pressure in the distribution system and are emptied on a hot day by means of large capacity pumps which take the water from the storage units into the distribution system. In this way, the peak demand on the filter plant, pumping stations, and large feeder mains is reduced very materially.

In addition to installing the storage units, which are used only in the summer, sprinkling restrictions have been in effect since 1948. There are 9 hr in each week between Jun. 1 and Sep. 15 when a consumer may not sprinkle his lawn. If he lives on one side of the street he may not sprinkle from 5 PM to 8 PM on Mondays, Wednesdays and Fridays. If he lives on the other side of the street he may not sprinkle from 5 PM. to 8 PM on Tuesdays, Thursdays, and Saturdays. Sprinkling at all other times is permitted.

By these mild sprinkling restrictions, peak demand in the high-service area has been cut from 3.8 times the yearly average to 2.5 times the yearly average, with the result that one-third of the plant capacity has been made available for expansion or other uses. There is no reduction in the total use of water for lawn sprinkling, because of these restrictions. People either sprinkle on the off-peak or do a more thorough job on their alloted evenings.

Pumping from the storage units has further reduced the demands on the major pumping stations so that on a peak day the highest demand on pumping stations and filter plant is twice, instead of three or four times, the yearly average. The saving in plant capacity is quite apparent.

A more complete description of Milwaukee's installations and experience is given in an earlier article by the author (*1*).

Reference

1. TANGHE, EDWARD F. Meeting Peak Demands at Milwaukee. *Jour. AWWA*, 47:60 (Jan. 1955).

Peak Demand Storage

George G. Schmid

A paper presented on Sep. 15, 1955, at the Michigan Section Meeting, Flint, Mich., by George G. Schmid, Engr.-Mgr., Southeastern Oakland County Water Authority, Royal Oak, Mich.

THERE are some water systems that can meet the peak hour on the peak day without apprehension. To many others, however, peak-hour demand is a challenge of considerable magnitude.

Some smaller communities whose population has remained static, or nearly so, and which possess favorable ground water supplies, have been able to meet peak loads by the use of auxiliary wells, operated during periods of peak demand. If the wells are advantageously located in the community they serve the purpose of storage, but cannot replace its effectiveness. Usually, therefore, a minimum amount of stored water, in either ground reservoirs or elevated tanks, is provided.

Other communities, because of population movement and industrial or other development and expansion, have had increased water consumption, and the extension of water use over a widespread area. Many of these face peak-hour problems, and to them peak demand storage has taken on a new significance.

Under some conditions, if supply and treatment are adequate, improvement can be made by the provision of additional mains and pumping equipment to meet peak loads, but such improvement must be carefully weighed against the possibility of accomplishing the same results by the use of storage designed and located to equalize the load on the distribution system.

Considering that the problem of peak-hour demand exists only for short periods of the year, and is often caused by uses (such as lawn sprinkling or air conditioning) which provide a limited amount of revenue over the year, it is clearly a challenge to decide what should be done to an existing system, and how to do it most economically, without unreasonable duplication of facilities. In any case, storage for peak-load demands deserves serious consideration.

Determining the amount of storage necessary to allow uniform pumping during the peak day and to meet the requirements of the peak hour requires a careful study of local conditions. Records of peak-day and peak-hour consumption provide the best basis for such a decision. If no such records exist, the next best practice is to make an approximation based upon the records or data available for the specific community or for nearby communities with similar problems. In this regard, caution is advisable, however, because hard and fast general rules cannot be applied to every situation. Each community has its own characteristics—topography, street layout, existing water main capacities and locations, sources of supply and treatment, density of population, neighborhood characteristics, and many other factors—which make it necessary that we develop and design storage around the

specific needs of the area or communities under consideration.

The system now being constructed by the Southeastern Oakland County Water Authority will be used as an example. Under this system treated water will be purchased from the city of Detroit and repumped through several booster stations into transmission mains of the Authority. From these the water will be metered into distribution mains of the several constituent cities. Both elevated and ground storage will be used. This system can be likened to that of an expanding suburban area containing a number of distribution systems which must be coordinated to function as a unit.

Design Specifications

In the design of the Authority system, the minimum storage capacity was based on the difference between the amount drawn from the Detroit water system (at a rate equal to the average demand for the peak day) and that required to meet the peak hours. The system, which is designed to fill all needs until 1970, will have instruments to record the peak hours; these records will later be used to adjust the original estimates.

Using a 1952 peak-hour day, the minimum controlled storage was determined to be approximately 26 per cent of the total daily requirement (estimated, at the start of operation, as 200 gallons per capita). The data used at this time were not complete, being composite data from seven communities and in various stages of compilation. Most of the communities had enforced sprinkling restrictions and it was never definitely known how severe the peak hour might be if unrestricted sprinkling (which was desired) were allowed. Later, additional records made in 1954 were used to estimate a peak day for 1956, at which time the entire system will be in operation. Table 1 was prepared on this basis and shows the estimated hourly consumption for the peak day. The differences between each hour's consumption and that of the average hour (in both percentage and gallons) and hourly cumulative totals are also shown.

As indicated in the table, the total estimated peak-day consumption for 1956 is 26.4 mil gal with an average hour of 1.1 mil gal, and a peak hour (8–9 PM) of 2.19 mil gal, almost double the average hourly consumption. Average-day consumption, for the entire year, was estimated at 12.96 mil gal, or 0.54 mil gal per hour; and for the summer months alone at 17.04 mil gal, or 0.71 mil gal per hour.

The booster stations and transmission mains of the system are designed to deliver a constant output over a 24-hr period based on the estimated peak-day rate for 1970, which is about 42.3 mgd, with sufficient storage, both elevated and ground, to provide the additional output required for the peak hours. The controlled storage capacity will provide the additional amount of water needed to meet the peak hours of the constituent municipalities while they are also drawing from the Authority system at a rate equal to the average demand for their peak days. The constituent municipalities will be responsible for improving their own distribution mains so that they are of sufficient capacity to distribute the water properly to local consumers.

Minimum Storage Needs

According to this design, the minimum storage required to meet the peak hour in 1956 is shown in Column 4, Table 1. The total of those figures

TABLE 1

Estimated Peak-Day Consumption, 1956

Time	Hourly Consumption *gal*	Hourly Consumption/ Avg Hr* %	Variation From Average Hour*—*gal*		Cumulative Consumption *gal*
			Above Average	Below Average	
6 AM	0	0			0
7	500,000	45.45	600,000		500,000
8	850,000	77.3	250,000		1,350,000
9	1,200,000	109.1		100,000	2,550,000
10	1,300,000	118.2		200,000	3,850,000
11	1,350,000	122.7		250,000	5,200,000
12	1,400,000	127.3		300,000	6,600,000
1 PM	1,450,000	131.8		350,000	8,050,000
2	1,500,000	136.4		400,000	9,550,000
3	1,650,000	150.0		550,000	11,200,000
4	1,600,000	145.4		500,000	12,800,000
5	1,400,000	127.3		300,000	14,200,000
6	1,600,000	145.4		500,000	15,800,000
7	1,800,000	163.6		700,000	17,600,000
8	2,000,000	181.8		900,000	19,600,000
9	2,190,000	199.1		1,090,000	21,790,000
10	1,400,000	127.3		300,000	23,190,000
11	800,000	72.7	300,000		23,990,000
12	550,000	50.0	550,000		25,450,000
1 AM	300,000	27.3	800,000		23,840,000
2	310,000	28.2	790,000		25,150,000
3	350,000	31.8	750,000		25,500,000
4	250,000	22.7	850,000		25,750,000
5	300,000	27.3	800,000		26,050,000
6	350,000	31.8	750,000		26,400,000
	26,400,000		6,440,000	6,440,000	

* Average hourly consumption was 1,100,000 gal.

which indicate hourly consumption in excess of the average is 6.44 mil gal, roughly 25 per cent of the total peak-day consumption of 26.4 mil gal. This percentage, which agrees very closely with that of 26 per cent established for the peak day in 1952, is represented by the shaded area in Fig. 1.

Another method of determining this volume has been described by John E. Kiker Jr., Professor of Civil Engineering, University of Florida, Gainesville, Fla. (*1*). Figure 2, which shows the amount of storage required using this method, agrees with the previous determination. In the preparation of Fig. 2, the accumulated totals shown in Column 6, Table 1, were used.

If, after the system is placed in service, it becomes evident that the amount of storage cannot meet the peak-hour needs, additional storage facilities will be provided. As indicated, the controlled storage requirement for the summer of 1956 will be about 6.44 mil gal. At this time, the Authority and its constituent municipalities will have 12.5 mil gal in storage, or about twice as much as the estimated demand. With such a margin, there should be

sufficient time to forecast any future needs. Additional storage, in the form of another 1.0-mil gal elevated tank, is planned for 1960—or before, if required.

Using the peak-day pumping rates to produce a uniform quantity of water and supplementary storage to meet the peak-hour needs was considered the best procedure for the operating conditions of the Authority system. The method of deduction used is not necessarily a general rule to be followed by others with like problems; other percentages or proportions may be necessary. The best results are obtained when the peculiar needs of the community in question are very carefully studied. Each system may have special characteristics or conditions which affect its peak-hour storage requirements; and what may be ideal for one community may be unsatisfactory for another.

Pumping-Rate Variations

To illustrate how a change in the pumping rate may affect storage requirements, Fig. 3 was prepared. This simple line graph shows the relationship between the percentage of storage required and the maximum rate that the system will produce under different pumping conditions. As previously stated, the Authority system pumping at 100 per cent of the peak-day rate will require about 25 per cent storage. If it were possible to pump at 140 per

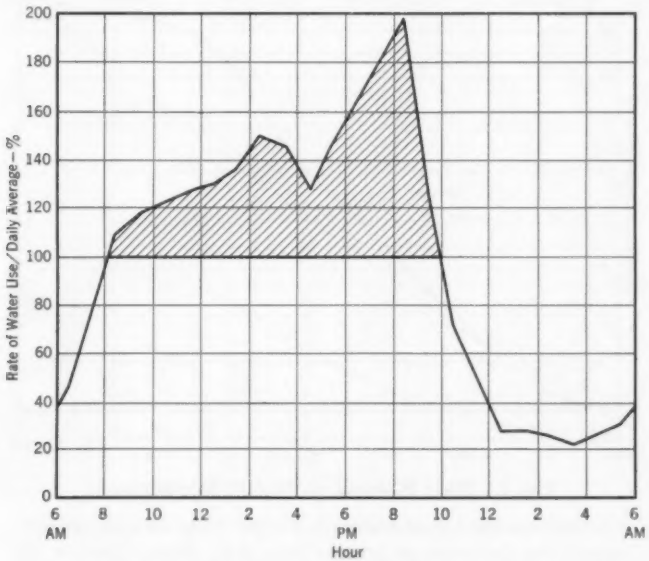

Fig. 1. Hourly Variations for Estimated Peak Day, 1956

The shaded area represents the volume of storage required to meet hourly peak demands, when pumping at the average hourly peak-day rate.

cent of the peak-day rate, for example, 6 per cent (or 1.6 mil gal) of storage would be needed; and if it were possible to pump at the average summer-day rate, which is about 65 per cent of the peak-day rate, then storage of about 46 per cent (or 12.13 mil gal) would be required. To go to an impossible extreme, if the plant were to pump at the average-day rate, which is about 49 per cent of the peak-day rate, storage of about 56 per cent, or 14.86 mil gal, would be required.

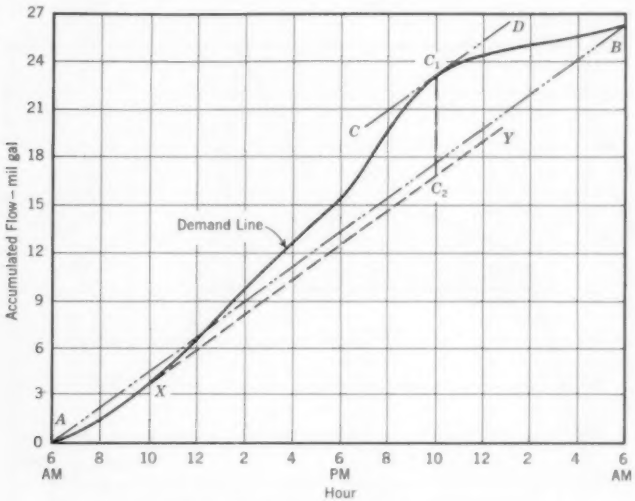

Fig. 2. Mass Diagram of Storage Requirements

The cumulative-demand curve is plotted from records or estimates, and the average-demand line, AB, *drawn between its extremities. Lines* CD *and* XY *are drawn parallel to Line* AB *and tangent to the curve at points of greatest divergence from the average. At* C_1*—the point of maximum divergence—a line is extended down the coordinate to Line* XY. *This line,* C_1C_2*, represents the required peak-hour storage: in this case, it scales to 6.44 mil gal.*

In the design of storage for the Authority system, existing local conditions were given highest consideration. The total storage capacity presently owned by the Authority (including its constituent municipalities) is 3.0 mil gal in elevated tanks. Now being added for use in 1956 is another 2.0 mil gal in elevated tanks and 10 mil gal in ground storage. If elevated storage is assumed to be 50 per cent effective in meeting peak-hour needs, then the total available amount of effective storage in 1956 will be 2.5 mil gal in elevated tanks and 10 mil gal in a ground reservoir, or 12.5 mil gal in all. When the additional 1.0-mil gal elevated tank is installed in 1960, the total storage capacity will be about 30.5 per cent of the design maximum-day (42.3) for 1970, which compares favor-

ably with the 25 per cent estimated for 1956. Of this total, the effective elevated storage of 3.0 mil gal represents 7 per cent, and the ground storage of 10.0 mil gal nearly 24 per cent, of the design maximum-day requirement of 42.3 mil gal.

To summarize, the total storage in the system is about 30.5 per cent of the designed maximum-day requirement; of this, 23 per cent is in elevated tanks, 77 per cent in ground storage. At present, it is believed that these proportions are reasonable, since they are based upon the best data available at the time of design. It should be remembered, however, that certain information (such as the actual severity of peak loads) was not obtainable. Sprinkling regulations in the several municipalities varied widely: in some, sprinkling was permitted on alternate days; in others, not at all. In spite of these factors, the design storage is considered adequate for both present and future needs, with some allowance for contingencies and variations from the original design data.

General Criteria

Some of the factors in the selection of the storage in the system described will be compared with the general considerations of storage. No specific rules or conditions will be defined, for each system has it own peculiarities and requires individual study. Determining the type and volume of storage best suited to a particular system depends as much upon practical considerations as it does upon the performance of routine calculations and assumptions based on rigid rules and formulas.

In the Authority system, the volume of storage was determined to be the

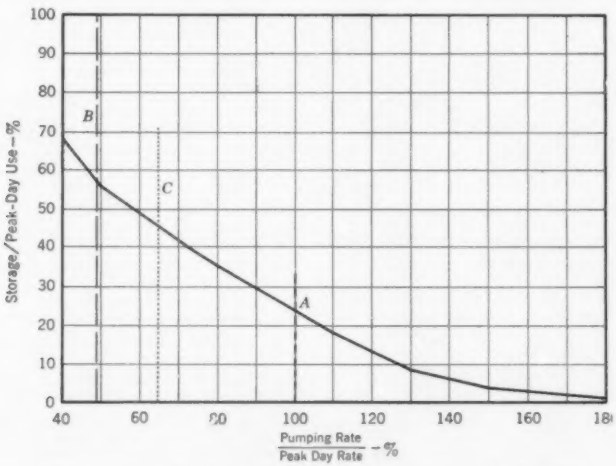

Fig. 3. Effect of Storage on Peak-Day Pumping Rate

Line A *represents peak-day rate, Line* B *the average-day rate (figured for the entire year), and Line* C *the average-day rate for the summer months only.*

additional amount of water required to meet the peak hours while drawing from the source at a rate equal to the average demand for the peak day. It is worth repeating that it would be naive to assume that the ratios and percentages used here could be employed as a general rule for other systems. The use of peak-hour storage to level peak-day demands, however, may have general application.

Storage in sufficient quantity provides a reserve for shutdown and fire fighting. It is a means of increasing the capacity and serviceability of water systems which local development and expansion have made inadequate. Properly located storage will reinforce the weak sections and level the load.

To produce at a greater pumping rate than the peak-day average may necessitate larger transmission and distribution mains and greater investment in booster stations and controls for limited use over the year. If a system with even a minimum of storage is supplied by mains that are not adequate to meet the demand of the peak hour, there will be a drop in pressure and additional pumping equipment will be required.

To produce at a lesser pumping rate than the peak-day average, on the other hand, requires costly additional storage. In the system described, had the production rate been made equal to the average-day requirement for the summer, the designed storage would have been 20 mil gal, or 7 mil gal additional. (This amount in ground storage would have cost $360,000.) Had the production rate been made higher than the peak day for the year, however, and the volume of storage decreased, it might have required day-to-day storage without reserve—an unsatisfactory condition.

A safe procedure, other factors being equal, is to determine the capacity of low-lift pumps and treatment facilities by the maximum-day demand, and the capacity of booster pumping and storage by the maximum-hour demand.

In the absence of good records, assumptions must be made. As a basis on which to proceed, several trial solutions, using anywhere from 20 to 50 per cent of the maximum peak-day demand, should give some indication of the volume of storage required.

Storage Location

Locating storage at the end of the line opposite the pumping facility, as well as at or near the area of peak demand, permits water to be fed from both ends of the system during periods of peak use, thus (in effect) increasing the carrying capacity of the mains. (It is assumed, in this case, that facilities will be available to permit filling reservoirs during low-consumption periods at night.)

Aesthetic considerations must also be given primary consideration in determining storage location. Elevated tanks in built-up residential areas may provoke local opposition, because of the height of the structure. Sufficient area should be obtained for the site to allow proper landscaping. In many cases it is possible to combine recreational areas —parks or playgrounds—with reservoir sites.

Reinforced-concrete storage reservoirs can be constructed partially below ground level, depending on local conditions. The large steel reservoir used by the Authority, though sufficiently exposed on the sides and top to permit maintenance work, is partially concealed by dikes and landscaping. The elevated tanks used are of the radial-cone type, one of them hav-

ing a fluted center, which gives it somewhat the appearance of a monument.

Although the storage used by the Authority comprises steel elevated tanks and a steel ground reservoir, consideration was previously given to other types of construction for the ground reservoir. The type selected, however, was considered the most satisfactory within the range of choice dictated by economic considerations.

system, fortunately, it was possible to combine additional pumping facilities for the ground storage with other pumping facilities, office space, and control equipment in the same building.

In the smaller community, the elevated tank can fill an important need. Always ready to serve with a reserve that does not require repumping, it is especially well suited, for example, for fire-fighting requirements. Usually,

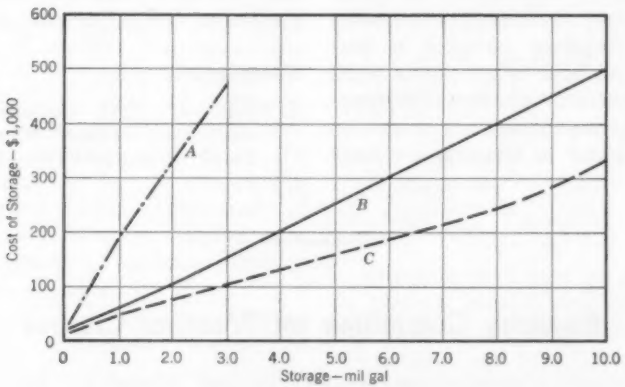

Fig. 4. Storage Cost Comparison

Line A *represents the cost of elevated storage, Line* B *the cost of ground storage (including pumping station), Line* C *the cost of ground storage without pumping station.*

Costs

The difference in first cost of elevated or ground storage can be compared by referring to Fig. 4, which applies to capacities up to and including 10.0 mil gal. The cost of steel elevated tanks up to 3.0 mil gal is compared with that of steel ground-storage reservoirs, including pumping stations. As a general rule, the cost of steel elevated tanks is about 1.5–2.5 times as great as that of steel ground reservoirs (including pumping stations) of comparable capacity. In the Authority

normal pumping operation is possible with its use. It is only during peak hours that elevated storage may be materially exhausted. The size of the tank should be determined by the pumping capacity of the station, but it should not be so large that it cannot be filled during the night hours of the peak day. In the larger community, a certain percentage of storage in elevated tanks is useful to maintain level pumping loads and permit more reliable pumping control.

Among the chief advantages of ground storage is that it can be filled

more rapidly than elevated tanks. With ground storage reservoirs in the smaller community, manual controls—requiring attendance at the site of operation—may be most suitable; in larger installations, however, the use of automatic controls may be preferred.

Either type of storage may be located near the area of peak demand and filled at night, permitting more efficient use of pumping equipment and water mains. Sometimes this practice may result in reduced power costs, since the required pumping is performed at an hour when power loads are at a minimum. The cost of repumping from ground reservoirs is also an important factor in selecting a storage facility. This cost, however, should not be excessive if repumping is required only for periods of peak consumption.

The type and volume of peak-hour storage best suited to any given system will be determined largely by the nature of the existing facility, and the special needs and conditions of the community served. Of the experience here described, at any rate, it may fairly be observed that peak-hour storage is to peak-hour demand as green is to grass.

Reference

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Advisory Committee on Weather Control

The advisory Committee on Weather Control, created on Aug. 13, 1953, was given responsibility for making "a complete study and evaluation of public and private experiments in weather control for the purpose of determining the extent to which the United States should experiment with, engage in, or regulate activities designed to control weather conditions."

Originally, a final report was to be presented to the President on Jun. 30, 1956, but, because of the need for further study, the committee has requested that the date for this final report be extended to Jun. 30, 1958.

The following accomplishments have been reported to date:

1. The committee has developed a superior methodology for evaluating weather control experiments statistically.

2. For the first time in history, an extensive series of evaluations has been completed showing that cloud seeding produces substantial and economically important increases in precipitation, at least in the Pacific Coast states.

3. The committee has initiated a vigorous program of physical evaluation which has already produced helpful results and gives assurance of providing information vital to the solution of the problems involved in weather modification.

High Strontium Content of Waukesha Water

Carl E. Moore and Otho McFarland

A contribution to the Journal by Carl E. Moore, Asst. Prof. of Chemistry, and Otho McFarland, Research Fellow, both of Loyola University, Chicago, Ill.

RECENT concern over the fallout of the radioactive fission products produced in nuclear explosions has prompted G. V. Alexander and coworkers (*1*) to make an extensive survey of the amount of natural, nonradioactive strontium present in the water provided for human consumption in the United States. Their study of approximately 50 water supplies indicates that the usual strontium content is well below 1 ppm.

During the course of recent studies of the analytical chemistry of strontium at Loyola University of Chicago it was called to the authors' attention that the public water supply of Waukesha, Wis., was considered to have a rather high strontium content (*2*). Samples were obtained and analyzed by flame photometry.* The most favorable wave length for analytical use was found to be 681 mμ, the peak intensity of the SrO band.

TABLE 1

Strontium Found in Waukesha Water

Well Location	Amount of Strontium *ppm*
Northtown	38
Baxter Street	24
Moreland	12

The analytical data, as summarized in Table 1, indicate that other municipal water supplies depending on wells or springs may have a much greater strontium content than that of the surface water sources of the larger metropolitan areas.

* Instruments used included a Model DU spectrophotometer with flame attachments and a 1P28 photomultiplier tube, products of Beckman Instruments, South Pasadena, Calif.

References

1. Alexander, G. V., Nusbaum, R. E., & MacDonald, N. S. *Jour. AWWA*, **46**:643 (Jul. 1954).
2. Lamar, W. L., Water Laboratory, US Geological Survey, Columbus, Ohio. Private communication.

California Experience With Plastic Pipe

Harold H. Yackey

A paper presented on Oct. 27, 1955, at the California Section Meeting, Sacramento, Calif., by Harold H. Yackey, Chief Engr., Suburban Water Systems, Puente, Calif.

PLASTIC materials have become increasingly commonplace in every phase of daily life. Because of its resistance to corrosion, lightness, ease of jointing, low cost, and excellent hydraulic-friction characteristics, plastic offers many possibilities in the water pipeline field. This article will discuss experiences with plastic tubing in general water utility use for 20 months in nearly 8,000 services

Operating Conditions

In the early spring of 1953, Suburban Water Systems, Puente, Calif., experimentally installed plastic tubing in place of copper. The plastic was used for water connections from the asbestos-cement distribution lines in streets to the meters in several small tracts. Plastic tubing with a diameter of 2 in. was also used in place of 4-in. asbestos-cement pipe for distribution lines in the dead-end streets in these same areas. Operating pressures in these tracts were 70–90 psi.

The results of the trial installations encouraged the general use of plastics throughout the various parts of Suburban Water Systems, which lie mostly in Los Angeles county, from Glendora on the north to south of Santa Ana Freeway, and from the San Gabriel River on the west to several miles east of the city of Covina.

From April 1953 until September 1954, approximately 7,740 plastic water services were installed. In making these installations, roughly 4,000 ft of ¾-in. and 88,000 ft of 1-in. plastic tubing was used. At the same time, 28,000 ft of 2-in. plastic tubing was installed in distribution lines.

During this period, plastic was installed in some sixteen separate units of the Suburban Water Systems. Some of these are operated as closed zones with well pumps working through hydropneumatic tanks, some are booster areas with booster pumps and hydropneumatic tanks fed from other supply sources (a large area near Whittier is supplied by a large gravity pipeline from the San Gabriel River), and still others are served by wells and boosters pumping to gravity storage tanks equipped with altitude valves.

Types and Weights

At the time of Suburban's installation, there were not many types of plastic available.

Polyethylene was the most commonly used material, because it was in general supply and was priced low enough to justify tests. While the recommended maximum pressures were lower than system requirements, the calculated burst pressures and ma-

terial test pressures were considered to provide ample margins of safety.

The polyethylene was available only as flexible pipe. This material, because it can not be cemented, welded, or similarly joined even to itself, requires the use of rigid fittings made of styrene or other material to which it is clamped like hose. In an irrigation system, use of such material may not be objectionable, but it is not a permanent enough method to be used for a domestic water system laid under pavement.

Saran, polyvinyl chloride, fibre glass–reinforced plastics, and other plastics were considered, but their costs were too high. Styrene and some other materials were suitably priced but were impractical for other reasons.

Butyrate,* at the time of the installations, was available in two types: a fairly flexible "copper tubing size" now called "solvent welded," and a rather rigid "iron pipe size." The latter was more expensive and reduced the cost advantage to too small a margin, but the "copper tubing size" was lower in price than actual copper and, for diameters up to 2 in., showed a saving even over galvanized iron. Although static pressures have increased since then, in 1953 most of the Suburban Systems were operating at less than 100 psi, and the butyrate tubing was recommended for pressures in excess of this.

The plastics decided upon then, in the ¾-in. and 1-in. sizes, were "solvent-welded" or "copper tubing weight" butyrate, with recommended working pressures of 118 and 119 psi, respectively, at 73.4°F. Most of the 2-in. tubing was of the same type and weight butyrate, with a recommended maximum working pressure of 107 psi. A small amount (3,600 ft) of "iron pipe size" polyethylene, with a recommended maximum working pressure of 52 psi, was also installed.

* Tenite II was used, a product of The Tennessee Eastman Co., Kingsport, Tenn.

Difficulties

Plastics shrink and expand much more than steel, even without excessive pressures.

The polyethylene failed rather quickly in the higher-pressure areas, therefore, and all of it has since been replaced. Failure was caused by stretching until the walls became thin and eventually blew out. Where the tubing did not enlarge, many tiny holes or series of short slits opened in the pipe. After stretching, the tubing stayed enlarged and repairs were nearly impossible to make. With the enlargement in diameter, there was an exaggerated shortening of length which caused joint failures. The pressures to which the polyethylene was subject, however, were excessive, and it should not be considered an inferior material because of its failure here.

The butyrate was troublesome in many ways, usually because of the excessive pressures it was subjected to. While some sections of the distribution systems operate under fairly light pressures, there are some sections that carry 120–125 psi, and some of the systems are subjected to rather heavy surges of pressure because of the starting and stopping of well and booster pumps. It was under these conditions that the butyrate gave the poorest performance.

The butyrate that was installed in pressure zones operating without surges and with less than 50 psi was almost as satisfactory as the similar copper installations had been, even though some early joint and fitting

trouble was experienced and a few pieces of tubing were cut or punctured during installation.

Materials of inferior quality caused many of the difficulties. Tubing from about six extruders and fittings from the same number of manufacturers were used, and it became apparent that these extruders and manufacturers did not maintain sufficient quality control over their products. The fittings and tubing varied so much in size that joints were either too loose or could not be made at all. Tubing that was the correct diameter for a fitting at the end of a full joint might not fit at all when the joint was cut. Some fittings failed due to weaknesses in design. Some plastic tubing became very brittle in a short time, a condition which showed up with buried material in service and also with stored tubing in warehouse stock. Other tubing developed countless small holes, even when not under excessive pressures. Some of the tubing tore longitudinally along rough lines, which seemed to indicate lack of uniformity in the material used. Many of these conditions did not apply to butyrate as a material, but only to various pieces of tubing. They were assumed, therefore, to be the result of manufacturing conditions.

Vendors were frequently overzealous about their products. It was very unfortunate that the plastics salesmen who were in the field at the beginning did not know their materials better. Recommendations as to installations vacillated back and forth. When fittings were tight, as in today's recommendations, pure solvent was recommended for the joints. When fittings were loose, a thickened cement of solvent and basic material was recommended. Some of their problems were caused by the manufacturers' lack of quality control.

Suburban Water Systems' own lack of experience caused many of the failures. Excess solvent, left at the edge of a fitting, weakened the tubing and, if the line was placed in service too soon, the tubing blew out. Insufficient expansion and contraction allowances in the trench caused the pulling apart of joints. It was finally found advisable to fill the lines and to put full pressure on the plastic before backfilling the trenches.

Very close supervision and tight control of operations was required, of course, when such new materials were used in so broad a program.

Tastes and Odors

After the plastics program was well under way at Suburban Water Systems, it was learned that, in other installations of butyrate, taste and odor problems were resulting. Special care was therefore given any complaints from these causes. With all the butyrate installations made during the 20-month period, not more than fifteen cases of difficulties with taste and odor developed. When they did occur, however, they were usually serious, and, with few exceptions, it was necessary to replace the plastic with other materials.

The taste and odor result because a form of chlorine added to the water attacks the plasticizer used in making the tubing. Better-quality water apparently does not use all the chlorine added, and, when this water gets into the butyrate pipe, trouble occurs. A small amount of such water caused trouble which sometimes disappeared eventually, but where the condition existed for several days, little success was

had in clearing the condition by methods available to the normal operating forces.

Polyethylene did not give any water quality trouble at all.

Unusual Results

Suburban's normal procedure, when 4-in. and 6-in. asbestos-cement distribution lines are employed, has been to use Class-150 pipe and tap the ¾-in. and 1-in. service connections directly into the asbestos cement. When copper is used for service connections, accidents to the service will frequently pull a piece out of the asbestos-cement pipe, requiring the replacement of an entire 13-ft pipe length. This does not happen with plastic, however, because the plastic is flexible enough to take the strain or, in some instances, to snap out of the asbestos cement without damage to the pipe. Even though the person or company causing the damage pays for the replacements necessary after such an accident, a repair bill of $25 results in much better public relations than a bill for $125, which would be required to replace an entire section of distribution main. Efforts are now being made to find a plastic connection piece for use between the asbestos cement and the copper.

Mention has previously been made of the marked expansion and contraction of plastic materials. A length of buried tubing moves enough to keep the soil away from itself so that when a leak occurs the water travels along the tubing. It is extremely difficult, therefore, to locate the leak. Since approximately half of the plastic lines installed are street crossings, it may be necessary to dig across an entire street or to replace the service with a bore every time a leak of this kind occurs.

Conclusions

After 20 months, Suburban Water Systems discontinued the use of plastics because the difficulties encountered indicated that the materials being used were not generally acceptable for the purpose. Use was also discontinued so that the maintenance requirements of the installed plastic could be determined.

The past year has seen a continuance of pressure failures in the higher-pressure and surge zones, and the butyrate was replaced in many such areas. In sections where the pressure has been increased, new weaknesses occurred, and when repeated leaks in the same location indicated continued trouble, the plastic was replaced.

It is estimated that, to date, approximately 10 per cent of the butyrate installations have given trouble and half these, or 5 per cent of total installations, have been replaced.

The Suburban Water Systems' basic reason for trying plastic was to reduce costs. The 2-in. plastic used was about half as expensive as 4-in. asbestos-cement. The way the services were set up, plastic-tubing installation cost about a third less than copper, the difference arising from the lower material cost. Asbestos cement and copper have since gone up in price—copper as much as 30–35 per cent. Plastic, however, costs about 30 per cent less than it did in 1953. The cost incentive for interest in plastics is, therefore, steadily increasing.

From Suburban Water Systems' experience with plastics, it is agreed that standard polyethylene was not suitable for the service to which it was subjected, and it should not have been used at all. The potential taste and odor problem, even though it was not too significant in the tests reported

here, would recommend against the further use of butyrate. There are many advantages to plastic pipe, and there will undoubtedly be plastics developed in the future which will have enough advantages and few enough disadvantages to warrant use.

Methods of testing plastic pipe are needed which will quickly show manufacturing weaknesses and will reveal those conditions which result in failures after the product has been placed in use.

The plastics vendors glibly guarantee the material against "rot, rust, or corrosion," but it would be better if they could guarantee against "deterioration." A typical quotation from a plastics manufacturer's catalog is the following: "While we believe this information to be accurate and reliable, no guarantee of its accuracy is made, and our product is sold without warranty as to fitness or performance and with the understanding that the purchasers will, by their own tests, determine the suitability of this product for their particular purposes."

Most of the difficulties experienced with plastics of Suburban Water Systems will eventually be avoided by control from the industry itself. The need for such control is already recognized, and the Society of the Plastics Industry has initiated a program which is expected to provide information, data, and advice to the fabricators of plastics that will set standards and controls for identifying water service pipe by branding it with the allowable working pressure, as is done with asbestos-cement pipe. In this way, definite warrantees and guarantees can be made.

Questions and Answers

There are a few questions which are repeatedly asked as to the performance of plastic tubing in water systems. These questions and their answers, in connection with experiences at Suburban Water Systems, follow.

What is the present financial status as a result of the use of plastics? The polyethylene was a complete loss. If it now costs twice as much to install the 4-in. asbestos-cement replacements in the completed tracts as it would have at the time of the polyethylene installation, the loss, with the polyethylene costs included, would be approximately $15,000–$18,000. Concerning the butyrate, even if all the butyrate which has failed—10 per cent of the total butyrate installation—has to be replaced, the saving on the balance will well justify the installation, polyethylene and all. The present problem is one of avoiding increasing pressure and surge strains so that unnecessary replacements may be avoided.

Would you recommend installation of butyrate similar to that used at Suburban Water Systems in a system where the pressures would not exceed 50 psi and where only unchlorinated well water would be used? No. Sooner or later chlorine would have to be used to clean up a line after repairs or in the installation of a new pipeline. Also, until the plastic industry has better control of the products being sold, there is no way of knowing what quality material will be received and what the deterioration of the material will be.

What is your own attitude toward plastics, and have you lost your enthusiasm? Definitely not! Every basic reason for the favorable consideration

of plastics is becoming more convincing. In addition to the improvements in plastics, the gathering control of the industry, and the spreading differential in cost between plastics and other materials, there is increasing difficulty in obtaining asbestos-cement pipe. Steel pipe is on a 120 day–delivery schedule, and the plumbers necessary for the installation of cast-iron pipe will be charging $3.525 per hour.

A product which would seem to merit some consideration is a very light-weight steel pipe lined and coated with plastic. Because of the low strength and cold-flow characteristics of plastic, larger sizes of all-plastic pipe will be too expensive, but a steel shell designed for pressure and backfill loading, with a plastic covering to give the advantages of plastic in contact with the water and for corrosion protection on the outside, would be worth some study.

The Public Utilities Commission is now preparing a new code for water systems. Do you think that plastics should be included or excluded as "approved material"? Any new code should not exclude plastics or any other material that may come up for trial use. In addition to those materials that the code may specifically approve, provision should be made for the use of any new or adapted material that merits consideration. It might be well to limit the extent to which such unspecifically approved materials could be used until a time when the AWWA, for instance, issued a standard specification for the material.

There are still quite a few cities and companies which very conscientiously refuse to accept asbestos-cement as an approved material. If the entire world were to take such an attitude toward new materials there would be no progress.

Discussion

W. T. Cruse

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The success which the Suburban Water Systems has had and is experiencing with its thermoplastic pipe installations demonstrates the promising future for this product of the plastics industry in water service. The failures which have been encountered make an uncompromising case for the necessity of great care in size control of plastic pipe and fittings. The experience underscores the importance of engineering within the rated performance of the pipe and operating within those limits. Moreover, it demonstrates the need for further research that will lead to standards which engineers may employ in designing systems. Laboratory research alone can never tell the full story. Both research and actual experience have contributed to developing information as to the performance ability of thermoplastic pipe, and they will continue to do so in the future.

Thermoplastic Pipe

This discussion will confine itself to thermoplastic pipe such as cellulose acetate butyrate, polyvinyl chloride, polyethylene, and copolymers of styrene.

Thermoplastic pipe was introduced in 1941, and since then its use in water services has continued to increase. The qualifications of thermoplastic pipe for such application were based on extensive fundamental material research. The materials were tested for impact, tensile, flexural, chemical, and other properties. This was essential procedure and the approach served an invaluable purpose.

As thermoplastic pipe came to be used for such diverse uses as water and gas transmission, in chemical and food processing plants, and by the oil industry, the need for more data on the physical properties and the need for standards became apparent. Projects have been inaugurated to obtain such data and to establish such standards.

Research

Three years ago research was undertaken at the National Sanitation Foundation of the School of Public Health, University of Michigan, Ann Arbor, Mich. The research was intended to establish the suitability of plastic pipe and its use in transporting potable water. This research is now complete. A program is being initiated whereby the right to use an "NSF" seal of approval will be given by the Foundation to the manufacturers who supply material for the thermoplastic pipe and to the extruders who produce the pipe. This procedure will enable the user and the public health officials who are concerned readily to identify "approved" thermoplastic pipe. The seal will attest to the fact that the pipe is suitable for use in services where drinking water is transported. This labeling program for pipe and fittings became effective Feb. 1, 1956.

Two years ago, research was initiated at the Battelle Memorial Institute, Columbus, Ohio, by the thermoplastic-pipe extruders and interested materials suppliers. This research was for the purpose of developing methods for testing the physical properties of thermoplastic pipe. The program has produced equipment for making burst tests and for exposing pipe to continued high pressures for long periods of time at controlled temperatures. This research contemplates the development of a method that will make possible a relatively short-term burst test to predict the long-time life of pipe. Plans are now being made to continue this research for another 2 years.

Standards

The following dimensional standards on thermoplastic pipe have either been established by the US Department of Commerce or have been approved by the Thermoplastics Pipe Division of the Society of the Plastics Industry, Inc. (SPI).

1. Dimensions and Tolerances for Flexible Standard-Wall Polyethylene Pipe (Commercial Standard 197).

2. Proposed Specification on Dimensions and Tolerances for 100-lb Pressure Flexible Polyethylene Pipe. This has been approved by the SPI Thermoplastics Pipe Division and is presently being circulated by US Department of Commerce.

3. Dimensional Standard on 75-lb Polyethylene Pipe. This proposed specification has been approved by the pipe and fittings manufacturers and has been forwarded to the US Department of Commerce for establishment as a voluntary commercial standard.

4. Proposed Commercial Standard for Dimensions and Tolerances for

Solvent-welded Butyrate Pipe. This has been approved by the Thermoplastics Pipe Division and will shortly be promulgated as a commercial standard by the US Department of Commerce.

5. Proposed Commercial Standard on Dimensions and Tolerances for Rigid Polyvinyl Chloride Pipe. This proposed specification has been approved by the SPI Thermoplastics Pipe Division and is in the hands of the US Department of Commerce where it will shortly be established as a voluntary commercial standard.

6. Proposed Dimensional Specification on Saran Pipe. This was approved by the SPI Thermoplastics Pipe Division and is presently under consideration by the US Department of Commerce.

Interassociation Activities

The following associations have fostered interest in industry-sponsored standards and have encouraged the Thermoplastics Pipe Division of SPI to use their facilities: American Gas Association, American Institute of Chemical Engineers, American Petroleum Institute, American Society of Mechanical Engineers, and AWWA.

The Thermoplastics Pipe Division of SPI has also moved to establish cooperative relations with the associations which play a significant role in developing the standards upon which industry relies.

SPI is acting as the sponsoring agency of a sectional committee, operating under the auspices of the ASA, on dimensions for plastic pipe. ASME is serving as a co-sponsor of this project. The sectional committee was formalized in late 1955 for the specific purpose of establishing, as American standards, the approved dimensional specifications of the SPI Thermoplastics Pipe Division.

A joint ASTM-SPI subcommittee on thermoplastic pipe and fittings has been established. The purpose of this subcommittee is to develop test methods appropriate for measuring the physical properties of thermoplastic pipe and to consider and establish dimensional standards on such pipe and fittings.

Use of Plastics

In this article, the author mentioned the importance of control over materials and pipe and fittings sizes. He also tells of the cost advantage of plastic pipe. Economically, thermoplastic-pipe installations are in a favorable competitive position. But, as water works engineers recognize, price is often an index of quality. The cheapest supplier is not necessarily the best, and this holds true for suppliers of plastic pipe as well as of any other item. As John Ruskin has said: "There is hardly anything in the world that some men cannot make a little worse and sell a little cheaper and the people who consider price only are that man's lawful prey."

In designing a water distribution system where iron, brass, copper, steel, concrete, or asbestos-cement pipe are specified, the engineers designate pipe sizes and fittings for a particular anticipated load, including a reasonable safety factor. In designing for plastic, the same considerations should be taken into account. Where a breakdown does occur because of overload, an environmental cause, or some other reason, the failure is attributed to the pipe, to incorrect dimension specifica-

tions, or to excessive operational loads, whichever is found to be the cause. When failures occur involving thermoplastic pipe, they should be evaluated in the same manner.

Tastes and Odors

The author commented on tastes and odors encountered in the cellulose acetate butyrate pipe installed in the Suburban Water Systems. Findings of the National Sanitation Foundation throw light on this subject, because this class of pipe was included in its test program. A taste-odor panel evaluated water circulated under controlled conditions through all pipe subjected to test in the project. The findings of the research are set forth in the following summary:

"None of the work on odor and flavor indicates that any of the types of plastic pipe used are likely to impart significant odors or flavors to water in an ordinary, regularly used distribution system, with the exception of a newly assembled system of plastic pipe in which highly volatile solvents or adhesives have been used in making joints which have not been aired thoroughly and flushed thoroughly before testing for odor and flavor."

Conclusion

A vast amount of research has been devoted toward developing thermoplastic pipe. Thousands of miles of pipe have been installed and is giving good service. The combination of research and experience which are building up daily is contributing to the accumulation of data and knowledge respecting the suitability of thermoplastic pipe for water service.

Instrumentation for Radioactive-Pollution Studies

Task Group Report

A report of Task Group 2630 P—Instrumentation and Methods of Testing Radioactive Contamination in Water, presented on Jun. 14, 1955, at the Annual Conference, Chicago, Ill., by Harold E. Pearson, Chairman, Research Chem., Metropolitan Water Dist. of Southern California, La Verne, Calif. The other members of the Task Group are H. H. Gerstein, Harry Hayes, Roy J. Morton, H. Gladys Swope, Frank S. Taylor, William F. Bale, Robert L. Butenhoff, and L. R. Setter.

A PRACTICAL understanding of the methods for detecting and measuring waterborne radioactivity is one of the immediate needs of the water works profession. With more widespread distribution of radioactive materials being constantly reported in industrial applications, in research, and for medical treatment, attention must be given to the increasing possibility of pollution of water supplies by radioactive wastes. Foremost among the causes for concern is the development of nuclear power. Although the radioactivity associated with radioisotopes and nuclear power is potentially of great benefit to mankind, it presents certain potential hazards which are becoming more and more commonplace and of increasing significance as concerns public health. The radiation hazards associated with the disposal of radioactive wastes are not insurmountable, but, to insure safety for the public, adequate instruments and testing procedures for the detection of radiation are necessary.

In the first report of Task Group 2630 P, published in 1952 (*1*), four methods for detecting radioactive contamination in water were described in detail. Two of the three nuclear instruments specifically recommended for making radioactivity measurements were primarily useful for testing the potability of drinking water in the emergency-level range suggested by the Federal Civil Defense Administration (*2*). The third instrument, an internal proportional counter, was recommended for separately assaying the gross alpha and beta radioactivity in water ranging from extremely low natural activity to very high activity caused by contamination.

The second committee report (*3*) reviewed those papers which had appeared in the *Journal* and in *Nucleonics* since the initial report and recommended that a survey be made to determine the types of instruments currently used and the sensitivity of the low-level measurements being made by the various water works laboratories.

To implement this survey, a questionnaire was mailed in October 1954 to 50 state and territorial health department laboratories and to 124 public and private utilities serving cities or special districts with populations over 100,000 requesting information on their program for making radioactivity measurements on water, the instruments available, and the methods employed. As of Mar. 31, 1955, a response was received from 42 of the state and territorial health departments and 66 of the public and private utilities.

The results of the survey are here presented, together with an outline of recommended instrumentation and methods for making low-level radioactivity measurements on water.

Survey Results

Most of the state and territorial health laboratories replied to the questionnaire, but almost half of the water utilities contacted failed to reply. It is probably safe to assume that most of the organizations which did not respond have no instruments or program of this kind underway. Replies to the questionnaire are summarized in Tables 1–5. Several organizations that replied negatively to the questions expressed a willingness to initiate a program as soon as the Association or the US Public Health Service made an official recommendation on instrumentation and procedures.

Of the 21 water utilities reporting some program or means of testing for radioactivity in water, twelve take water samples for testing regularly, sixteen indicate that they have their own instruments for making either low-level or emergency measurements, three send the samples to state laboratories, and two report that the work is, or will be, done by city health departments.

Whenever a completed questionnaire indicated that an outside agency was or would be making the radioactivity measurements, a copy of the questionnaire was sent to that agency, and the instruments available were included in the tabulation. Occasionally the agency that the water utility was dependent upon for making radioactivity measurements was also lacking in proper instrumentation. These are extremely unfortunate situations! There may be a few cases, however, not reported to the Task Group, where city health groups are equipped with radiation detection instruments about

TABLE 1

Survey of Instrumentation for Measurement of Radioactivity in Water

	State and Territorial Laboratories	Public and Private Utilities
Number of questionnaires mailed	50	124
Number of replies	42	66
Number of organizations reporting:		
No instruments for radioactivity measurements and no sampling program	19	45
Regular program for making radioactivity measurements on water	7	12
Frequency of measurements		
Continuous	1	3
Weekly		3
Monthly		6
Variable	6	
Occasional radioactivity measurements made and prepared for emergency testing	7	9
Tests made or to be made by reporting agency	14	16
Tests made or to be made by outside agency	2*	5†
Ownership of any type of radiation detection instruments (water, air, personnel protection)	23‡	19§
Operators of instruments receive some special training	13	10

* Occasional samples sent to Robert A. Taft San. Eng. Center, US Public Health Service, Cincinnati, Ohio.
† Three cities send samples to state laboratories, two report work done by city health departments.
‡ Does not include complete set of instruments reported by Robert A. Taft San. Eng. Center.
§ Includes those instruments owned by two city health departments mentioned under †.

which the water department is unaware. The 21 water utilities mentioned previously are those reporting an actual program with instruments either owned or available through an outside agency.

Twenty-three state and territorial laboratories reported that some type of radiation detection instrument was owned, but several reported that the instruments available were intended for personnel protection or occupational health surveys, and were not for water analysis. Fourteen public health laboratories stated that the instruments owned were for making either emergency or low-level radioactivity assays on water; six had instruments suitable for both purposes. Only eleven were adequately equipped for low-level counting, and three others only for checking emergency tolerance levels.

TABLE 2

Survey of Instrumentation Types and Number of Radiological Instruments in Water Laboratories

Type of Instrument	State and Territorial Laboratories*		Public and Private Utilities†	
	Number With Instruments	Total Number of Instruments	Number With Instruments	Total Number of Instruments
A —Geiger-Muller survey meter	17	28	11	15
B—Ion chamber	18	58	6	66
C—Electroscope or electrometer	11	40	13	19
D—Geiger-Muller scaler	9	13	4	4
E—Proportional counter	5	7	4	4
F—Scintillation counter	4	5	1	1
G—Continuous-monitoring equipment	2	3	4	5
H—Others	9	55	3	10
One or more of any type	23		19	

* Does not include complete set of instruments reported by Robert A. Taft San. Eng. Center.
† Includes those instruments owned by two city health departments mentioned in note †, Table 1.

Types of Instruments

The number of organizations reporting ownership of various types of nuclear instruments is tabulated in Table 2. Some of the reporting groups did not report the exact number of instruments owned, but indicated "several" or "many." The data on number of instruments owned, therefore, are useful principally for comparison of the numbers in each category that are available. Not included in these statistics, is a complete set of all these instruments reported by the Robert A. Taft Sanitary Engineering Center, US Public Health Service, Cincinnati, Ohio. The instruments at this laboratory were not included because the number of each type was nearly the same as the total number reported by all of the state health laboratories.

Type A

The Geiger-Muller survey meter measures hard beta plus gamma with the shield open, and gamma only with the shield closed. It is normally calibrated in three ranges: 0–0.2 milliroentgens per hour (mr/hr), 0–2 mr/hr, and 0–20 mr/hr. By Method 1 in the original Task Group report (*1*) and the Federal Civil Defense Administration bulletin (*4*), it can be used

for making an assay of radioactivity in water within the range of emergency tolerance levels. It is also useful for environmental surveys for occupational health protection. In Table 3 it may be noted that water works operators are planning to use Geiger-Muller survey meters for both these purposes, but nine of the state laboratories intend to use them solely for personnel protection.

The portable, thin side-wall tube, beta-gammas survey meter shown in Fig. 1 reads in milliroentgens per hour with three full-scale ranges of 0.2, 2 and 20 mr/hr. Each instrument must be calibrated against suitable standards equivalent to known activities in water. The marked influence of beta energy on the position of the calibration curve has recently been demonstrated by Lacy and Kahn (*5*). Thallium (Tl^{204}), a radioisotope proposed as a standard by Wheler, Reynolds, and Brooksbank (*6*), because its maximum energy is approximately the same as the average beta energy for fission products, gives a slightly lower meter reading than fission products having equivalent activity. Thus, standardizing against Tl^{204} would tend to err on the conservative side in estimating the activity present.

The lower limit of sensitivity for emergency assay of a 150-ml water sample varies from 0.01 to 0.1 microcuries (μc) per liter depending on the radioisotope present. The upper limit for an undiluted sample is above the permissible emergency tolerance level of 90 μc per liter. It is obvious that the range of sensitivity can be somewhat extended either by concentrating or diluting the sample. To keep the geometry and self-absorption factors constant, a uniform volume of sample is presented to the counter tube in the same relative position at all times.

The various models of Geiger-Muller survey meters reported by all state and water utility laboratories are shown in Table 4. The cost ranges from \$140 to \$280. In general, greater stability and reproducibility of results may be expected from the medium to higher-priced models.

TABLE 3

Intended Application for Instruments Listed Under Types A, B, and C

Application	State and Territorial Laboratories With Instruments			Public and Private Utilities With Instruments		
	Type A	Type B	Type C	Type A	Type B	Type C
For emergency radioassay of water	2	2	5	5	1	13
For personnel protection monitoring	9	13	3	0	5	0
For both radioassay of water and personnel protection monitoring	6	3	3	5	0	0

Type B

The ion chamber instruments are primarily useful for environmental surveys to detect gamma radiation from radioactive sources, which may constitute a health hazard, and X-ray. Some also measure beta radiation. There are wide variations in the ranges of ion chamber instruments, depending on application. Most state laboratories are equipped with ion chambers for detecting radioactive contamination, but

only six of the water utilities have any of this type.

Predicted fallout patterns for atomic bombs, especially the more powerful thermonuclear weapons, indicate that widespread contamination may be expected whenever earth, building materials, or water are engulfed in the fireball. Water departments in areas vulnerable to enemy attack should give serious consideration to providing instruments of this type for safeguarding personnel who may be assigned to make repairs or to operate equipment in a contaminated area. With proper instruments and training, these personnel will be aware of the safe stay-time and thus will not exceed a maximum permissible exposure.

The radiation hazard due to fallout in the area surrounding a reservoir during the early disaster phases is likely to be a more serious problem than the contamination of the water stored there. This statement is based on theoretical considerations of self-absorption of radiation by water, the dilution of activity caused by the depth of reservoirs, and the tendency for radioactive particles to settle to the bottom of the reservoir because of their low solubility. If radiation readings were sufficiently high to require evacuation of an area, the water might be seriously contaminated, but it is not likely that the latter will occur.

The total number of ion-chamber survey instruments owned by state and water department laboratories is greater than that for any other type of instrument, but they are principally in the hands of a few organizations. The operating range of ion-chamber instruments is quite variable, depending on the service requirements. The cost may vary from $170 to $400.

Type C

An electroscope, or electrometer, is an ionization chamber with a gold-leaf or quartz fiber indicating-electrometer. Electroscopes must be calibrated against known isotopes. The electroscope was recommended for the measurement of emergency levels of radioactive contamination in water as described under Method 2 in the original report (*1*). Thirteen water utilities and eleven state laboratories have acquired this type of instrument, which is illustrated in Fig. 1.

Sensitivity of the electroscope is dependent upon the energy of the radiation and the amount of ionization caused by each kind of radioactive emanation present. It also depends on the method of sample preparation. In general, laboratories reported sensitivities as low as 0.03–0.1 μc per liter for gross beta-gamma and beta activity on 150 ml of liquid sample. Limiting sensitivity for gamma activity is about 1–20 μc per liter. On an evaporated sample, 6.5 disintegrations per minute of alpha activity can be readily measured because of the higher ionizing power of alpha particles; thus, 100–1,000 $\mu\mu$c per liter of alpha can be detected. Lacy and Kahn (*5*), who have determined electroscope sensitivity to several radioisotopes of different energy characteristics, state that minimum concentrations (in microcuries per milliliter) detectable with the Landsverk L-75 are of the order of 10^{-6} for alpha, 10^{-4} for beta, and 10^{-2} for gamma radiation (0.001, 0.1 and 10 μc per liter, respectively).

The suggested maximum permissible concentration of fission products in drinking water for use during a 10-day period after an atomic explosion is 0.09 μc per milliliter or 90 μc per liter. This value is well above the minimum

detectable concentration for these electroscopes.

The models of electroscopes now owned by water laboratories are listed in Table 4. The prices range from $100 to $200, with accessories.

The principal advantages of survey meters and electroscopes are low cost and simplicity of operation. A minimum of training is required for operators to learn their use. It was pointed out by Setter (*1*), however, that considerably more training is necessary in order to understand the limitations of these instruments and the significance of readings. Lack of an appreciation of these limits of sensitivity may lead to a false sense of security regarding pollution by specific radioisotopes resulting from waste disposal by expanding nuclear industry.

For low-level counting—that is, for measuring the traces of naturally-occurring radioactive elements in water and for detecting contamination introduced from peacetime uses of radioactive materials—Geiger-Muller scalers (Type D) using thin-window Geiger tubes or proportional counters (Type E) are required. Instruments such as these are considerably more expensive than those already discussed, and few water laboratories are equipped with them.

Type D

Nine state laboratories and four water department laboratories own precision Geiger-Muller scaler equipment for radioassay. This equipment, illustrated in Fig. 2, uses a sensitive, thin end-window, Geiger-Muller counter tube (usually mounted in a lead shield containing a shelf support to hold sample dishes at one or more fixed distances below the counter tube), and the scaler unit, which contains a stabilized high-voltage power supply, electronic amplifiers, and output circuits to trigger an electronic-counting system. The counter responds to individual pulses caused when an ionizing particle enters the detecting Geiger tube. This type of instrument is useful primarily

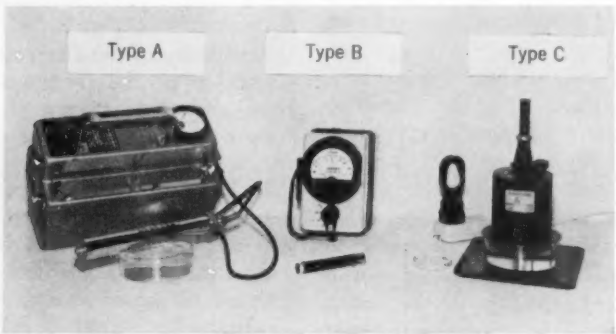

Fig. 1. Types A, B, and C Instruments

From left to right, the instruments are: Type A—Beckman MX-5 Geiger-Muller survey meter; Type B—Jordan Radector AG-500 ion chamber; and Type C—Landsverk L-75 electrometer. At center of the photograph is shown an L-28 pocket dosimeter.

for the measurement of beta radiation. Alpha radiation is absorbed before entering the tube either by the air between the sample and the thin window or by the window material. Gamma radiation can also be detected, but at much lower efficiency than beta—0.5–1.0 per cent for counters with light-element cathodes.

The dried residue obtained by evaporating a large sample of water in a small dish is placed under the tube for a suitable counting time. The methods employed by the reporting laboratories and the lower limit of sensitivity indicated for the measurements are shown in Table 5. Factors affecting sensitivity of counting low-level activity will be discussed after describing the remaining types of instruments.

The various models of Geiger-Muller scaler equipment owned by water laboratories cost from $700 to $1,000 with accessories. Some laboratories reported automatic sample changer and data-recording devices which may be obtained at additional cost.

Type E

The internal proportional counter was recommended for precise measurement of low-level waterborne radioactivity in the original Task Group report (*1*). Five state laboratories and four public utilities own nuclear instruments of this type. Proportional-counter systems require a counting chamber into which the sample is placed, a flow of gas through the chamber to displace the air introduced when changing samples, a stabilized power supply, electronic amplifiers, pulse discriminators, and output circuits to operate an electronic scaler (*see* Fig. 2 and 3). By selecting a low voltage in the proportional region with suitable amplification of pulses, the larger pulses produced by ionizing alpha radiation can be separated from the smaller beta and gamma pulses. Thus, it is possible to count alpha disintegrations separately from the beta and gamma. When the voltage is raised into the Geiger region, the pulse caused by beta and gamma ionization increases sufficiently to count these emanations also.

Samples are prepared for counting by evaporating portions of water on planchets. The size of the sample counted by the various laboratories is noted in Table 5. Sensitivity for alpha counting is greater than for beta counting because of the extremely low natural alpha activity—normally less than 0.1 count per minute. Proportional-flow counters are not usually surrounded by a lead shield, and the natural background due to cosmic radiation, therefore, is approximately 50 counts per minute at the beta-gamma counting voltage. This raises the limit on beta sensitivity, as noted in Table 5.

The several proportional counter systems available in water laboratories at the time of the survey are listed in Table 4. The cost of these instruments, with accessories, varies from $1,000 to $1,500. Compressed bottled gas, an argon-methane mixture (90–100 per cent), or methane alone, is needed to flush the counting chamber. With each change of sample, there is a time delay because of the flushing necessary to remove air from the chamber. Small amounts of residual air or water vapor have a pronounced effect on the beta-counting rate (*7*), and several minutes and considerable gas may be wasted before it is certain that the chamber is completely flushed. If a large number of samples are to be counted, a preflush flow counter (*see* Fig. 3) eliminates this delay and saves considerable gas. A rotating support with three wells for the planchets per-

TABLE 4

*Models and Manufacturers of Instruments Owned by Reporting Laboratories**

Model	Manufacturer
A—Geiger-Muller Survey Meter	
Beckman MX-5	Beckman Instrument Co. Fullerton, Calif.
El Tronics SM-3	El-Tronics, Inc. Philadelphia, Pa.
Model GLR 200A	Herbach and Rademan Co. Philadelphia, Pa.
Keleket K-800	Kelley-Koett Instrument Co. Cincinnati, Ohio
Lucky Strike	Lucky Strike Precision Instrument Co. Denver, Colo.
Nuclear Model 2610A Nuclear Model 2611 (count rate meter)	Nuclear Instrument and Chemical Corp. Chicago, Ill.
Professional 107	Precision Radiation Instrument Co. Los Angeles, Calif.
Tracerlab SU-3B Tracerlab SU-5A	Tracerlab, Inc. Boston, Mass.
Victoreen 389	Victoreen Instrument Co. Cleveland, Ohio
B. Ion Chambers	
Beckman MX, 5 ranges: 0–20, 0–50, 0–200, 0–2,000 mr/hr	Beckman Instrument Co. Fullerton, Calif.
El Tronics S 1D-1	El-Tronics, Inc. Philadelphia, Pa.
Jordan Radectors, 2 ranges AG-500: 0–500 mr/hr, 0–500 r/hr AG-50: 0–50 mr/hr, 0–50 r/hr AG-15: 0–15 mr/hr, 0–15 r/hr	Jordan Electronics Co. Pasadena, Calif.
Juno Ion Chamber SA501	Technical Assocs. Glendale, Calif.
Keleket K352, 3 ranges, 0–50, 0–500, 0–5,000 mr/hr	Kelley-Koett Instrument Co. Cincinnati, Ohio
Samson Alpha Survey Meter Model D-5 0–2,500, 0–12,500 cpm.	Radioactive Products, Inc. Detroit, Mich.
Tracerlab SU-1F "Cutie Pie" 0–25, 0–250, 0–2,500 mr/hr Tracerlab SU-10A	Tracerlab, Inc. Boston, Mass.
Victoreen 247-A, 4 ranges: 0–2.5, 0–25, 0–250, 0–2,500 mr/hr Victoreen 70, available in various ranges from 1 mr to 1,000 roentgen total radiation	Victoreen Instrument Co. Cleveland, Ohio

* No listing in this table shall be construed as a recommendation of the Task Group.

TABLE 4 (contd.)—*Models and Manufacturers of Instruments**

Model	Manufacturer
Type C—Electroscopes or Electrometers	
Landsverk L-75	Landsverk Electrometer Co. Glendale, Calif.
Landsverk Radicond L-76	
Lauritzen Electroscope	F. C. Henson Co. Pasadena, Calif.
Lind Electroscope	
Type D—Geiger-Muller Scalers	
Atomlab A-72	Atomlab, Inc. New York, N.Y.
Berkeley Scientific 1000B	Berkeley Div., Beckman Instrument Co. Richmond, Calif.
Nuclear Lab. Model 163	Nuclear Instrument and Chemical Corp. Chicago, Ill.
Nuclear Model 172	
Model RCD-1	Nucleonic Corp. of America Brooklyn, N.Y.
Omniometer, Mark 9, Model 15	Radiation Counter Lab., Inc. Chicago, Ill.
Tracerlab SC-1B	Tracerlab, Inc. Boston, Mass.
Tracerlab 64 Scaler	
Type E—Internal Proportional Counters	
Model PC-1. $2\frac{1}{4}$-in. diam. internal-flow counter complete with power supply and scaler	Nuclear Measurements Corp. Indianapolis, Ind.
Model PC-2B. $2\frac{1}{4}$-in. diam. internal-flow counter, complete with power supply and scaler	Nuclear Measurements Corp. Indianapolis, Ind.
Model PCC-10 Converter. $2\frac{1}{4}$-in. diam. flow counter with amplifier and discriminator circuit	Nuclear Measurements Corp. Indianapolis, Ind.
Mark 12, Model 2, Preflush Flow Counter. $1\frac{13}{32}$-in. diam. with Mark 9 Model 4 Nucleometer	Radiation Counter Lab., Inc. Chicago, Ill.
Mark 12, Model 3, Preflush Flow Counter. 2-in. diam., used with NRDLD 12 Scaler	Radiation Counter Lab., Inc. Nuclear Radiation Development Lab. Chicago, Ill.
Type F—Scintillation Counters	
General Electric Counter. SC-5, SC-10	Nuclear Research and Development, Inc. St. Louis, Mo.

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TABLE 4 (contd.)—*Models and Manufacturers of Instruments**

Model	Manufacturer
Type F—Scintillation Counters (contd.)	
Ken Research Geigerscope	Ken Research, Inc. River Edge, N.J.
Model CAX-15 Alpha Counter	R-C Scientific Instrument Co. Playa del Rey, Calif.
Tracerlab P-12 Alpha Probe	Tracerlab, Inc. Boston, Mass.
Type G—Continuous-monitoring Equipment	
El-Tronics RM4B count rate meter with Esterline Angus 24-hr recording chart AW	El-Tronics, Inc. Philadelphia, Pa.
Mark 15 Model 10 count rate meter with Esterline Angus recorder	Radiation Counter Lab., Inc. Chicago, Ill.
Mark 9, Model 10 continuous recording system	Radiation Counter Lab., Inc. Chicago, Ill.
Type H—Miscellaneous (Pocket Dosimeters)	
Beckman Model 102, 0–200 mr Model 103 Charger	Beckman Instrument Co. Fullerton, Calif.
Keleket K-700, 0–200 mr K425 charger	Kelley-Koett Instrument Co. Cincinnati, Ohio
Landsverk L-28, 0–200 mr L-24 charger	Landsverk Electrometer Glendale, Calif.
Tracerlab SU-8H, 0–200 mr SU-9 charger	Tracerlab, Inc. Boston, Mass.
Victoreen R-Condenser Meter laboratory monitor	Victoreen Instrument Co. Cleveland, Ohio

* No listing in this table shall be construed as a recommendation of the Task Group.

mits a sample to be counted in one well as the exhaust gas is sweeping out any air and water vapor from another sample in the next well. When the base is revolved 120 degrees, the preflushed sample is ready for counting. The third well is open for introducing and removing samples to keep the cycle continuous.

Type F

Scintillation counters (*see* Fig. 4), with an activated-zinc sulfide phosphor, a photomultiplier tube, and an associated scaler, have a high efficiency for detecting alpha particles but do not record beta or gamma disintegrations Substitution of phosphors, in conjunction with additional amplification of the pulse received by the photomultiplier tube, permits the measurement of beta or gamma radiation. Four reporting water laboratories are equipped with alpha scintillation counters for low-level radioassay work, and one has a portable gamma scintillation counter containing a thallium-activated sodium iodide crystal detector.

The alpha scintillation counter can be operated to give a yield of 25–35 per cent, depending on the permissible background noise (*8*). Maximum geometrical efficiency of counting is 50 per cent, and the above yield, therefore,

represents relatively good detection efficiency. Alpha scintillation counters can be used with most of the electronic-scaler and power supply units designed for Geiger counting simply by connecting the scintillation counter to the unit and adjusting to the proper operating voltage. The alpha scintillation counters cost from $175 to $300. If a scaler discussed under Type D is not available it must be provided at an additional cost of $600–1,000.

Type G

Four water utilities reported that the water supply was continuously monitored by using a Geiger counter tube attached to an integrating count rate meter with a 24-hr recording chart, as shown in Fig. 5. One state laboratory continuously monitors radioactivity in air and another indicates that water is monitored continuously but does not state the method employed.

At one treatment plant a mica end-window Geiger counter is suspended by means of a float in a natural sedimentation basin. The count rate meter readings are then recorded on a 24-hr recorder chart. At another, water is passed continuously through a jacketed glass Geiger counter tube and counts per minute indicated on a rate-counter dial are recorded on a strip chart. The monitor may be equipped with an alarm circuit which will trip an alarm relay when the activity exceeds a preset rate.

The instruments are calibrated against known concentrations of P^{32}, I^{131}, or mixed fission products in water. The lower limit of sensitivity indicated by calibration charts and statements submitted by the reporting agencies is in the range of 0.01–0.1 μc per liter. As noted in an earlier discussion on liquid samples, the self-absorption by water decreases the amount of radiation reaching the detection chamber.

The cost of continuous monitoring equipment is approximately $1,500. Until the sensitivity can be materially improved, it appears that the use of this type of equipment will be limited to critical areas where the hazards of radioactive pollution, caused by radiological warfare, sabotage, or spillage of atomic wastes, are very great.

Type H

Miscellaneous nuclear instruments, such as pocket dosimeters (*see* Fig. 1) and laboratory monitoring devices were reported under this heading. The statements made about Type B instruments concerning their usefulness for personnel protection also applies to Type H instruments. The devices described under Type B are rate meters, whereas the dosimeter registers the accumulated radiation exposure. As time goes on, water and sewage works operators may need these protective instruments.

Training of Technicians

Some special training is desirable for the operators of instruments of Types C, D, E, F, and G. To properly evaluate the data obtained, the operator must appreciate the characteristics of the electronic equipment which affect its reliability and must be aware of the factors influencing the efficiency of counting the particular radiation present in the sample. Thirteen state laboratories and ten water utilities report that their technicians have received special training. More than half of these agencies—nine state laboratories and three utilities—state that their operators have received special training at the Robert A. Taft Sanitary Engineering Center. Many more water departments should take advan-

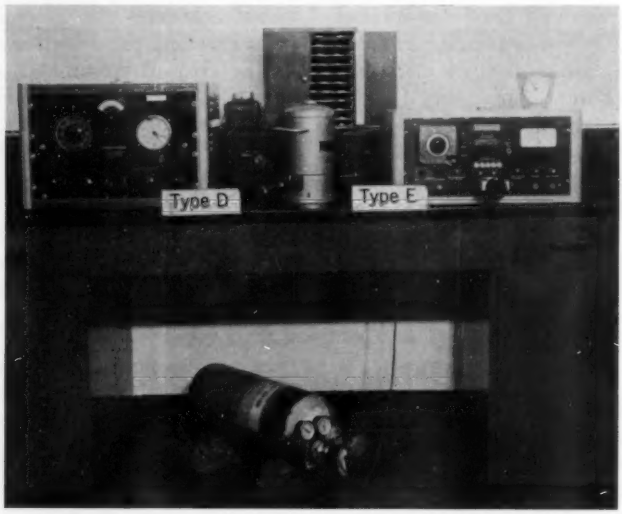

Fig. 2. Examples of Types D and E Instruments

The instrument at left, Type D, is a Tracerlab, Inc., Autoscaler SC-1B, with SC-9D shielded sample changer and a Geiger-Muller counter. The Type E instrument at right is a Nuclear Measurements Corp. PC-2B proportional counter.

tage of the excellent training given at the Center. Information on course schedules may be obtained by writing to the Radiological Health Training Section, Robert A. Taft Sanitary Engineering Center, Cincinnati 2, Ohio.

Sensitivity Comparisons

While discussing the several types of nuclear instruments, various limits on the minimum amount of radioactivity that could be detected by each method of measurement were mentioned. It was noted that the sensitivity of the electrometers and survey meters depended on the energy of the activity present. To compare the sensitivity of the several types, however, a single calibration curve for each has been drawn in Fig. 6. Because the calibration curve for a Geiger-Muller survey meter, plotting readings in milliroentgens per hour against activity, follows a pattern similar to the curve for Type C of Fig. 6 for the Landsverk unit, and lies in the same general area, it is not shown.

Radioactivity levels to the right of the points where the curves intercept the background count rate or the drift rate for the quartz fiber electroscope are detectable. In the region between the background rate and twice background, the precision of the readings is considerably reduced, particularly for the Type G integrating rate meters which respond quickly to the random disintegration rate. Through the use of lengthy counting periods, the minimum sensitivity for the Geiger-Muller scaler and the proportional counter can be extended to a net sample count rate considerably below the background rate.

TABLE 5

Sensitivity of Low-Level Radioactivity Measurements Reported by Water Laboratories

Preparation of Sample and Counting Procedure	Sensitivity—$\mu\mu$c/liter			
	Gross	Alpha	Beta	Gamma
Type D—G-M Scaler				
Evaporated 250 ml water to dryness	100			
Evaporated 200 ml or less to dryness	20	no	20	20
Evaporated 1 liter, count 10 min	1,000	no	1,000	no
Evaporated 500 ml in 1-in. steel planchet		no	6−9±4	
Evaporated 1 liter in 1-in. steel planchet, count twice for 32 min		no	4−10±5	
Evaporated 5 ml to dryness		no activity detected		
Activity in 500 ml flocculated with 10 ppm $FeCl_3$, filtered, ashed, and transferred to counting dish.			1,000	
Type E—Internal Proportional Counter				
Filtered 250 ml thru millipore filter. Evaporated 250-ml duplicate samples in 2-in. Al dish, alpha and beta counted separately on suspended solids and dissolved salts, each 32 min		1	10	
Evaporated 500 ml, count about 20 min	1,000	10	1,000	
Gross alpha and beta separately on 2-in. steel planchets				
Alpha—evaporate 20 ml, count 16 hr		1		
Beta—evaporate 200 ml, count 2 to 3 hr for beta, 32 min for alpha		1	3–10±5	
Evaporated 500 ml on 1-in. planchet	3	0.01	3	10

It can readily be observed that the survey meters and Landsverk unit are sensitive to activities far below the emergency values of 90 μc per liter and well below the maximum permissible level of certain specific isotopes noted at the bottom of Fig. 6—that is, Na^{24}, Cl^{36}, Ca^{45}, and Fe^{55} (*9*). The maximum permissible concentration of a number of other isotopes is considerably below the range of sensitivity of the survey meter or the electrometer. A few of these—Sr^{89}, Sr^{90}, U (natural), Pu^{239}, and Ra^{226}—are also shown. Any one of these radioactive elements and some others listed in Table 3 of a US Bureau of Standards handbook (*9*) could be present in harmful amounts and yet go undetected by the emergency monitoring instruments.

In the handling of industrial radioactive wastes, it would be extremely hazardous to allow the pollution caused by specific isotopes to approach the maximum permissible concentration because fluctuations in flow rates, biological activity, and other concentrating processes may produce a hazardous condition. Healy (*10*) states that maximum permissible levels for public water supplies are considered as 1–10 per cent of the values in the US Bureau of Standards handbook. Foster and Rostenbach, in a discussion of an article (*11*), reported that radioactivity levels of alpha and beta emitters in

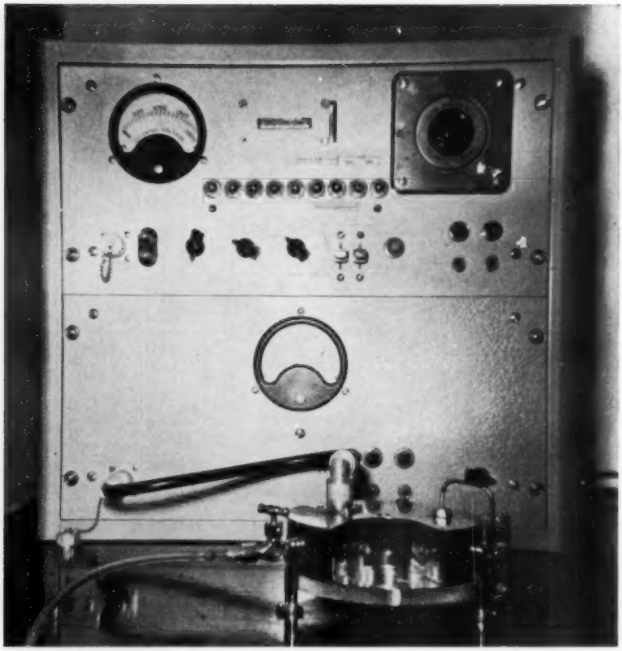

Fig. 3. Proportional Counter

The instrument shown is a Radiation Counter Labs., Inc., Mark 12, Model 2, preflush proportional counter of $1\frac{1}{2}$-in. diameter, with a Mark 9, Model 4, nucleometer.

the Columbia River, downstream from Richmond, 20 miles below the Hanford atomic energy plant, are well below one-tenth of the recommended maximum permissible concentration (*9*). If safety factors of this order are imposed, and these appear minimal, regulating agencies should be able to detect one hundredth to one thousandth of the maximum permissible level of any radioisotope which may be present in the water.

These restrictions on pollution would shift the maximum permissible concentration of radioactive Na^{24} to 0.08–0.8 μc per liter. This is below the sensitivity of the electroscope and survey instruments. A reactor-cooling water containing 100 ppm of sodium will have Na^{24} formed in excess of 0.8 μc per liter when exposed to a neutron flux of 10^{12} per square centimeter for 3 sec. The neutron bombardment will produce other radioisotopes from the elements present in the water and radioactive corrosion products may be washed from heat-exchanger surfaces. Consequently, as reactor operations spread, the need for accurate appraisal of radiation levels is becoming more acute.

The continuous monitoring ratemeter occupies a position intermediate between the emergency-monitoring survey instruments and the low-level assaying Geiger and proportional coun-

ters. Hazardous concentrations of some radioisotopes might slip by the continuous monitor unnoticed, and, to avoid being lulled into a false feeling of security, an occasional analysis of concentrated liquid samples, or preferably of the dried residue obtained by evaporation, would be necessary.

The reason for the low sensitivity of measurements with liquid samples is brought out forcefully by Lacy and Kahn (*5*), who showed that in the measurement of a P^{32} solution, in a 200-ml dish under the L-75 electroscope, the value of the reading was not altered appreciably in increasing the volume from 60 ml to 150 ml. In this particular liquid sample, the radiation coming from more than 0.3 in. below the surface was being absorbed by the water. This is known as self-absorption.

The factors affecting the overall efficiency of counting include geometry, window and air absorption, back-scatter, self-absorption, and self-scatter. Self-absorption and self-scatter are primarily determined by the nature of the sample. For the highest detection efficiency, it is desirable that the geometry and back-scatter factors be large, and that the window-absorption and self-absorption losses be small.

The physical geometry of counting with an end-window Geiger-Muller tube is only 5–15 per cent of the true radioactive-disintegration rate, depending on the distance between sample and window. For samples containing only traces of radioactivity the precision is poor, compared to internal counters where the physical geometry of counting is 50 per cent.

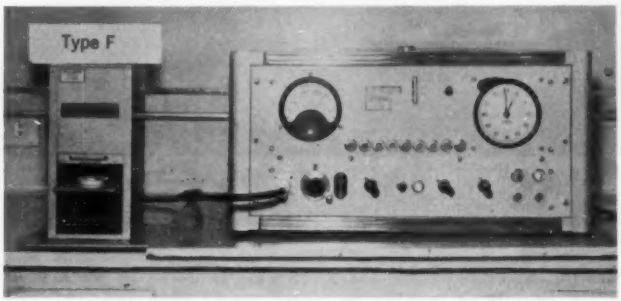

Fig. 4. Type F Scintillation Counter

An R-C Scientific Instrument Co. Model CAX 15 alpha scintillation counter is shown, with Radiation Instrument Development Lab. Model 512C scaler.

Beta activity of weak energy absorbed by the air and window of the Geiger-Muller tube is not counted, and appropriate correction for air and window absorption is extremely difficult for unknown mixtures of radioactive elements. The internal counter eliminates these losses and, therefore, the necessity for such corrections.

Back-scatter of beta radiation improves the overall yield for Geiger-Muller counting to a limited degree, related to the beta energy emitted. With the internal counter, backscatter of beta radiation can raise the beta yield to 60–75 per cent of the total disintegrations.

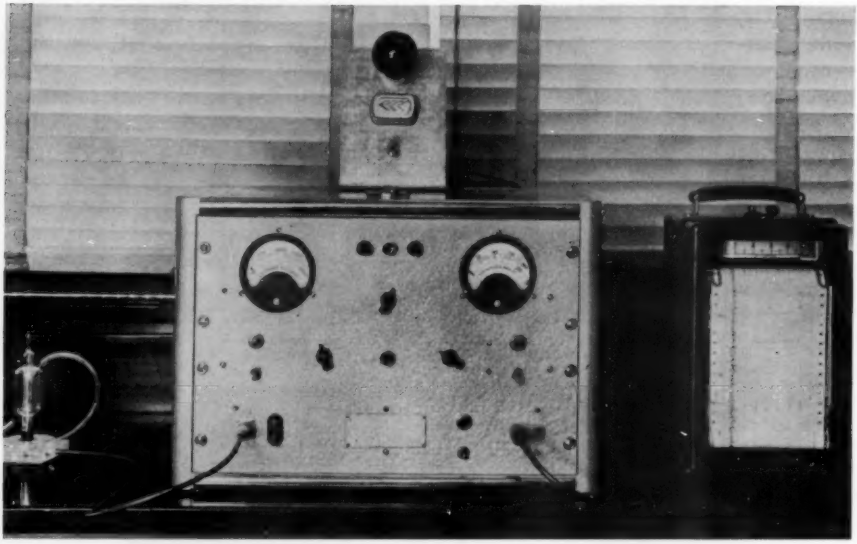

Fig. 5. Type G Continuous Monitoring Equipment

A Radiation Counter Labs., Inc., Mark 15, Model 10, continuous monitor is shown.

Self-absorption by the nonradioactive salts present in the dried residue reduces the counting efficiency for either counting system. Figure 7 shows the effect of sample thickness on counting alpha and beta radiation from natural uranium, the principal radioactive trace element occurring in Colorado River water. The limitation on thickness of sample is more critical in the case of alpha radiation than beta radiation. In addition to the Nuclear Measurements PC-2B proportional counter, which holds a 2-in. diameter dish, the Metropolitan Water District of Southern California owns a Tracerlab SC-1B scaler, which takes a 1-in. diameter dish. The smaller dish has one-fourth the area of the larger dish, increasing the thickness fourfold for samples of equal size. Because of the higher yield of the proportional counter and the lower losses due to self-absorption, a 200-ml sample can be assayed for gross beta activity by the internal proportional counter with greater precision than a 1-liter sample can be assayed by the Geiger-Muller scaler.

Similar comparisons of the relative sensitivity of the two methods for making low-level radioactivity measurements have been reported by Graf, Comar, and Whitney (*12*). In a series of papers, Setter and his co-workers (*13–15*) have evaluated the factors influencing the performance of the proportional counter. Their study of the effect of the statistically random nature of radioactive disintegrations on the lower limits of detectability should be studied by anyone interested in making radioactivity measurements.

Recommendations

Delineating the responsibilities of the individual water utility or public health agency for measuring radio-

activity in water is not a function of this Task Group. There is a great need for more basic knowledge on natural levels of radioactivity in waters in order that the incidence of pollution may be recognized. For those organizations interested in making either emergency or low-level radioactivity measurements, the following recommendations are submitted.

Emergency Radioactivity Assays

Methods 1 and 2, proposed in the original Task Group (*1*), continue to have merit as emergency monitoring procedures. Operators, however, must be cognizant of the limits of sensitivity based on a calibration curve. Radioactivity measurements which do not exceed background shall be reported as "less than x μc per liter in terms of uranium oxide" or whatever standard was used. Reporting "no activity present" may be misleading.

In addition to calibration against prepared standards, as recommended by Hursh, Zizzo and Dahl (*4, 16*), periodic calibration against liquid standards containing radioactive thallium, Tl^{204}, would be desirable (*5, 6*).

In the use of the Landsverk analysis unit, the operator is cautioned that the amount of ionization caused by alpha, beta, and gamma radiation is roughly in the ratio of $10^4:10^2:1$. The quartz fiber drift rate should be observed three times—with sample uncovered to detect gross ionization, with sample covered with paper to absorb alpha, and with sample covered with thin metal shields to absorb alpha and beta radiation. Details of these operations may be obtained from the manuals which accompany the instruments.

Low-Level Radioactivity Assays

The internal proportional counter is recommended for separately assaying the gross alpha and beta radioactivity of waters and industrial wastes. For samples containing more than 200 ppm of dissolved solids, an internal counter for large 2-in. diameter dishes is desirable for minimizing the losses caused by self-absorption. Except for allowing greater latitude in the selection of counting time, the procedure is the same as that recommended by Setter, Goldin, and Nader (*15*), which is as follows:

The suspended matter in a 200–1,000-ml sample is removed by filtration through a membrane filter or, at some sacrifice to efficiency, through a smooth, hard-surfaced filter paper. The suspended matter of the membrane is oven dried at 103°C, cooled, and counted. To obtain the sample thickness for self-absorption correction, the weight of suspended solids is determined by *Standard Methods* technique (*17*).

A sample of filtrate (200–1,000 ml) preferably containing less than 100 mg of dissolved solids, is evaporated directly in a tared metal dish using infra-red heat lamps or bottom heat with the temperature of evaporation controlled not to exceed 90°C, thus avoiding splattering. The sample is then oven dried at 103°C, cooled, weighed, and counted. The volume of sample may be reduced when the radioactivity present in the sample is considerably above background levels. For example, a 10-ml sample of water containing the maximum permissible concentration of Sr^{89} (0.07 μc per liter) will give about 1,000 beta counts per minute.

The alpha and beta operating voltages are chosen at the middle of the alpha and beta plateaus, respectively. The alpha plateau may be determined by using an uncovered uranium oxide source, 5 cm in diameter, or larger.

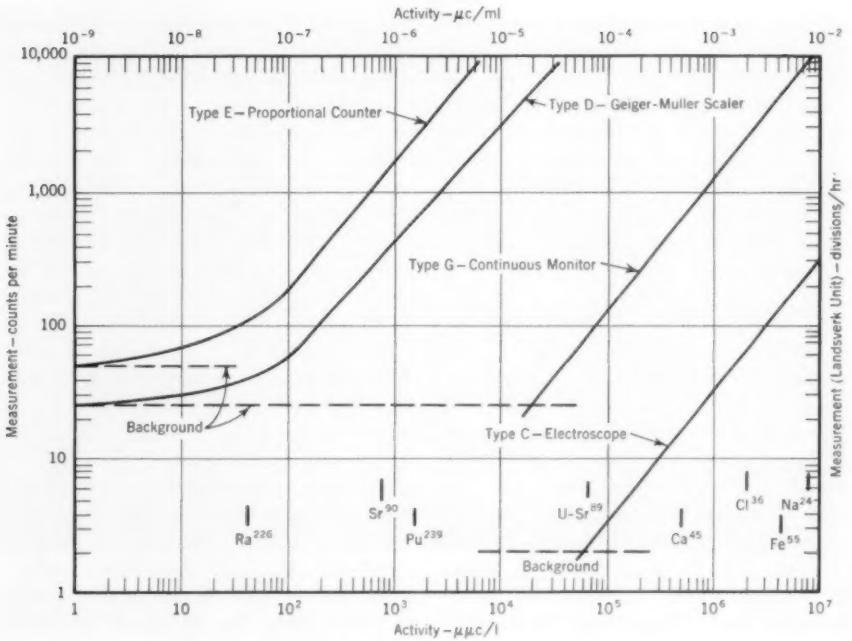

Fig. 6. Comparison of Sensitivity of Instruments

The Type C instrument was a Landsverk L-75 electroscope, and the Landsverk units are given at right of the figure. The sample for the Type C instrument was a 150-ml liquid sample; for Type D and Type E instruments, 1-liter dried sample; and for Type G instrument, a liquid sample. The Type G instrument was a Geiger-Muller ratemeter.

Standards of Po^{210} or RaD-E-F (Po^{210} in equilibrium with Pb^{210} and Bi^{210}) can be used as alpha sources in place of uranium oxide. For determining the beta plateau and establishing the beta yield of the counter, a uranium oxide source covered with aluminum foil (10 mg per square centimeter) is suitable. Thallium (Tl^{204}) and Sr^{90} sources may also be used as beta standards. When reporting results the calibrating standards should be stated.

A daily check on instrument reliability is made by counting a standard source three times at the voltage required for the planned assay work. Background activity is determined by counting clean sample dishes three times a day at the proper voltage. When alpha and beta assays are to be made, standardization and background measurements must be made at both operating voltages.

Samples are counted for two equal periods separated by a short flushing time. If the two counting rates for the sample do not agree within counting statistics, a third count is taken immediately. A minimum counting time of 32 min per sample will give a net count above the detectable limit for many waters. To obtain more precise results in laboratories where relatively few samples are counted, however, a longer counting period may be practical and desirable.

When the alpha activity is less than 5 counts per minute, the observed beta count may be corrected by subtracting the alpha count. When the alpha activity equals or exceeds 5 counts per minute, the sample is covered with aluminum foil (7 mg per square centimeter in thickness) and the observed beta count is corrected for aluminum-foil absorption by extrapolating an absorption curve (*18*) to zero thickness. Because the counter efficiency and sample volume are known, the counting rate can be converted to micromicrocuries per liter by the formula:

$$\mu\mu c/l = \frac{1{,}000 \times \text{net sample count (counts per min)}}{2.22 \times \text{vol. counted in ml} \times \text{yield factor}}$$

Net sample count rate is equal to the gross count rate of sample plus background minus the background count rate. The yield factor is determined from the standard sources; the yield for alpha activity is usually 50–52 per cent and, for beta, 50–75 per cent, depending on the amount of back-scatter.

Standard Nomenclature

The adoption of a standardized nomenclature for reporting results is definitely needed. Because of the exceedingly broad range of activities encountered, the exponential notation has been widely employed—for example, 4×10^{-8} and 8×10^{-3} μc/ml. Water works operators usually report results in parts per million or milligrams per liter. It is recommended that two forms of expression be used to report the amount of radioactivity present in water: micromicrocuries per liter ($\mu\mu$c/l) for activities at and near the amounts usually found in water, and microcuries per liter (μc/l), for high-level activity due to disposal of nuclear-

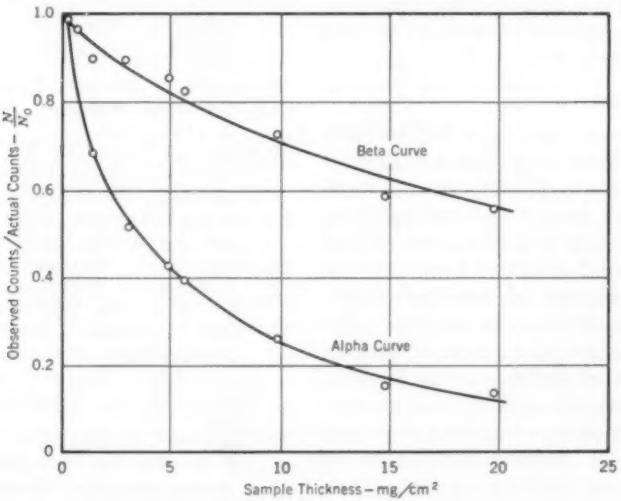

Fig. 7. Effects of Self-Absorption on Counting Alpha and Beta From Natural Uranium

A Nuclear Measurements Corp. PC-2B counter was used on a sample of Colorado River water spiked with U (natural) and evaporated on 2-in. stainless-steel planchet.

industry wastes. In the above example, the values become 40 μμc/l and 8 μc/l. The spread of values anticipated can be covered in the ranges of 0.01–10,000 μμc/l and 0.01–10,000 μc/l. Since the symbol μ for micro is equivalent to one one-millionth, a reading of 10,000 μμc/l is equal to 0.01 μc/l. Except for occasional cross-references to other notations, the recommended terminology has been employed throughout the text of this report.

Conclusions

The present survey of instrumentation for radioactive-pollution studies indicates that only eleven state and territorial health laboratories and only nine water utility laboratories have trained personnel and adequate equipment for the measurement of low-level waterborne radioactive contamination. The need for extended interest in this aspect of water quality control is great.

To encourage more active participation in this phase of modern water examination, a method for measuring low-level radioactivity in water employing the internal proportional counter is recommended and described.

Acknowledgments

For the generous cooperation of all personnel associated with the water utilities and state and territorial health laboratories who supplied the data reported in this survey, grateful acknowledgment is made.

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Cathodic Protection at Los Angeles

W. E. Kirkendall

A contribution to the Journal by W. E. Kirkendall, Corrosion Tester, Corrosion Section, Water Distr. Div., Dept. of Water and Power, Los Angeles, Calif.

CATHODIC protection has been recognized for some time as one of the more effective, practical, and economical methods for mitigating corrosion of the interior of steel water tanks. Unfortunately, the technical literature available up to the present time has been largely concerned with theoretical concepts and does not give enough practical information to guide the water works operator in the correct design and installation of cathodic-protection systems for water tanks. At the time the work described here was started, completely reliable data could not be found which would supply a practical working knowledge of the voltages and currents at which protection could be obtained without damaging the high-dielectric protective coatings commonly used on the walls of water storage tanks.

The above considerations led the Los Angeles Department of Water and Power to undertake an extensive testing and practical-experimentation program for purposes of developing working knowledge and techniques necessary to the successful practice of cathodic protection. This has resulted in the realization of practical methods of measurement and installation design whereby the protective-current requirements for any water tank may be determined quite accurately and the installation design matched to these current requirements. The purpose of this article is to describe in some detail the procedures and methods employed to obtain engineering data needed for economical installation, operation, and maintenance of cathodic-protection systems for water tanks.

Background

Cathodic protection, both for bare-steel (uncoated) tanks and for tanks well coated with coal-tar enamel, has now been successfully used for over 5 years in the Los Angeles water system and the results have proved to be outstandingly economical. Greatest economy has been realized where the tanks received primary protection from properly applied coatings of coal-tar enamel and supplementary protection from magnesium anodes.

Experience with cathodic protection in the Los Angeles water system has been confined to waters having the analyses shown in Table 1. Columns *A–D* of this table show analyses of waters supplied from *A*—surface streams in the mountain areas; *B*—well-water sources; *C*—the Los Angeles River; and *D*—Colorado River sources after filtration and softening. Some of the cathodically protected tanks contain mixtures of these waters in various proportions. Water works operators within the areas of the Pacific Southwest and Rocky Mountain regions will undoubtedly recognize characteristics of their own water supplies in the analyses given under one or more of the columns. It should also

be understood that the method found to be ideal for preventing corrosion of water tanks in the mild climate of Southern California might have to be slightly modified when extended to climates in which freezing conditions are experienced.

protective coatings from the ferrous metal base. Specifically, tank-to-water potentials of up to 1.5 v, as measured with a copper sulfate reference electrode, were found to be safe when coatings of coal-tar enamel were used. At excessively high voltages, currents may

TABLE 1

Analyses of Typical Domestic Water Supplies *

Item	Amount—*ppm*†			
	A Owens Valley Aqueduct Water	*B* Van Owen Well Water	*C* Los Angeles River Water	*D* Colorado River Water
Total dissolved solids	202	366	489	663
Total hardness ($CaCO_3$)	84	216	276	127
Alkalinity ($CaCO_3$)	115	164	192	102
Sulfates (SO_4)	24	101	140	283
Chlorides (Cl)	19	20	39	84
Nitrates (NO_3)	0.3	7.5	13	0.2
Calcium (Ca)	25	61	79	31
Magnesium (Mg)	5.3	15	19	12
Sodium (Na)	35	36	55	176
Potassium (K)	4.1	2.8	3.7	3.4
Silica (SiO_2)	20	22	24	10
Iron (Fe)	0.1	0.0	0.0	0.0
Aluminum (Al)	0.02	0.02	0.02	0.02
Manganese (Mn)	0.01	0.01	0.01	0.01
Boron	0.53	0.14	0.19	0.15
Fluoride (F)	0.6	0.3	0.4	0.4
pH	8.50	7.75	7.65	8.76
Portion of total supply—%	79.3	6.2	8.9	4.0
Spec. cond.—$K \times 10^6$	324	570	737	1060
Avg. temp.‡—°*F*	59	65	68	63

* Values shown are averages for fiscal year 1951–52. A 25 per cent deviation from average may be expected in values during the year.
† Unless otherwise noted.
‡ Temperatures are for water entering distribution system. Temperatures varied from 43°F to 84°F, depending on time of year.

Scale-Model Experiments

Scale-model experimental tanks were constructed for the determination of practical operating criteria and other pertinent data. In these tanks it was shown that cathodic-protection currents, which produced tank-wall potentials in excess of certain critical values, would cause deterioration, blistering, and progressive separation of organic

pass through the coating at such points of weakness as pinholes and ruptures, carrying along water and aggressive ions by a process known as electro-osmosis. This brings about an accumulation of water and gas beneath the coating which separates it from the metal in localized areas. When the additional bare metal is exposed by this process, the current required for pro-

tection may increase and, if protective-current flow becomes inadequate, deep pits may result in the steel even though such sites still appear to have good protective-coating coverage. These pits may occur at any time and may progress unnoticed until failure of the tanks.

Because of the breakage of the treated graphite rod anodes used in early installations and because this breakage became known only at times of periodic inspection, thus making the down-time highly uncertain, it became evident that more rugged anodes were desirable. Accordingly, both aluminum and magnesium anodes were considered and tested in the scale-model tanks. These tests indicated that the most economical performance could be obtained with magnesium anodes used in conjunction with coal-tar enamel coatings. The fact that magnesium does not require an external source of power indicated that substantial economies could be effected by eliminating costs of rectifiers and installation of a-c power lines and accessory equipment. Especially significant was the fact that magnesium anodes could be used in conjunction with coal-tar enamel without damage to this coating. Aluminum anodes, although somewhat cheaper than magnesium, did not prove effective in the tests for stopping corrosion except when powered by an external current supply. These tests led to the adoption of magnesium anodes for use in all cathodically protected tanks in the Los Angeles water system.

It should be of interest, especially to those who use water from cathodically protected tanks, that tests show no adverse effect on water quality due to metallic ions introduced by either aluminum or magnesium anodes. With raw city water flowing through the scale-model tanks, with rate-of-flow to tank-capacity ratio carefully adjusted to approximate that prevailing in standard operating tanks, and under the most adverse possible conditions of cathodic protection,* samples of water were taken from the inlet and outlet lines and analyzed with very good results, as shown in Table 2.

TABLE 2

Analyses of Waters From Scale-Model Experiments

Item	Amount—*ppm**			Increase Over Raw Water *ppm*
	Inlet Water	Outlet Water		
		Tank With Al Anodes	Tank With Mg Anodes	
Magnesium	24.00	24.00	26.00	2.00
Aluminum	0.07	1.70	0.07	1.63
Alkalinity	1.87	1.91	1.94	Slight
pH	7.9	8.15	8.23	Slight

* Unless otherwise noted.

These data show that only very small increases of metallic ions (dissolved aluminum and magnesium) were caused by the flow of cathodic-protection current from the anode

* The scale models were bare-steel tanks which received currents of 1.9 ma per square foot, which is several times the average current density flowing through similar quantities of water contained in standard tanks.

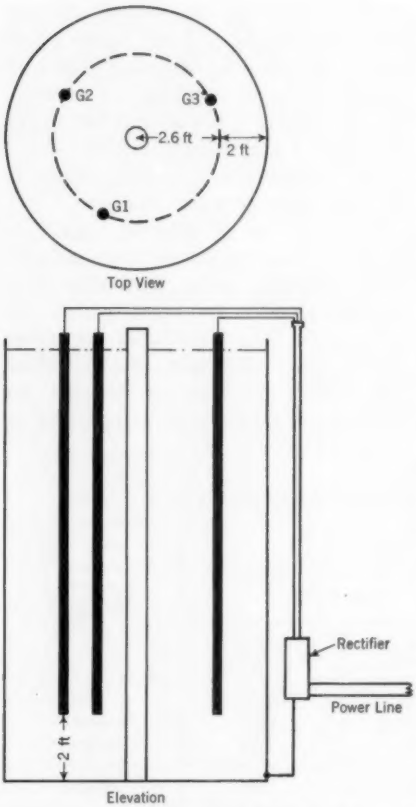

Fig. 1. Installation Design for Lakeside Tank

Three treated graphite anodes—G1, G2, and G3—were permanently installed in the tank and connected to a selenium cell rectifier.

through the water. These slight increases of aluminum and magnesium content were accompanied by correspondingly small increases of total alkalinity and pH.

Lakeside Tank

The first Los Angeles installation of cathodic protection in an operating tank was made in Lakeside tank during the first 6 months of 1942. Lakeside tank is small—approximately 10 ft in diameter by 16 ft high, mounted on an elevated structure to provide adequate pressure. At the time the cathodic protection was installed, the tank had a coating of enamel which had become extensively alligatored and was ruptured in many places, causing rust pits to develop beneath the coating. The water contained in Lakeside tank had approximately the analysis shown in Column *A* of Table 1.

When cathodic protection was first considered for Lakeside tank, treated graphite rods were the only suitable anode material available. It was necessary to avoid using material, such as the scrap iron anodes sometimes found in the cathodic protection of underground pipelines, because these would adversely affect or contaminate the domestic water supply. Because platinum anodes were very expensive, they could not be used.

After making a survey of current requirement and potential according to methods which will be described later, three treated graphite anodes were permanently installed in Lakeside tank and connected with a selenium cell rectifier to supply the needed cathodic-protection current. This installation is illustrated in Fig. 1. Between 2.2 and 2.7 amp of current were supplied by the rectifier, operating at a terminal voltage range of 6.8–7.3 v.

Tests made soon after installation showed that the efficiency of converting a-c energy supplied to the rectifier into d-c cathodic-protection output was 57.5 per cent. The major portion of the original installation cost of the cathodic-protection system in Lakeside tank was for the selenium cell rectifier and installation of the a-c power supply. The continuing monthly charges for the electricity which was used were a minor item.

Subsequent inspections disclosed that further corrosion and pitting had been completely arrested, and that the crevices and ruptures in the protective coating of enamel had become filled with a relatively dense, impervious, tightly adhering scale of high electric resistance. Analysis of this scale showed it to be largely calcium carbonate containing traces of magnesium compounds, iron oxides or hydroxides, and other substances.

While corrosion had been stopped, it was necessary to make periodic adjustments of the rectifier terminal voltage to avoid excessive accumulations of scale. This was because high rates of scale formation may result from an excessive flow of protective current, which, if continued, can be injurious to the protective coating of enamel. Some scale formation is desirable, however, because it acts to seal preexisting defects in the coating and thereby enables complete protection to be obtained with a minimum flow of protective current.

An additional maintenance problem resulted from the fact that the graphite rods used as anodes were brittle and occasionally one or more would break off and fall to the bottom of the tank. At such times it was necessary to use a line and hook or to drain the tank in order to recover, repair, and reinstall the anode. Because breakage became known only at times of scheduled inspections, further uncertainty was introduced as to the duration of the unprotected period.

Because of the difficulties encountered with the graphite anodes, and in view of the favorable results obtained with magnesium anodes in the scale-model tanks, the anodes in Lakeside tank were replaced with eight magnesium anodes during the latter part of October 1948.

The vertical magnesium anodes were extended down to within 1 ft of the bottom of the tank and semicircular anodes were installed around the tank at the bottom, close to the tank wall. Since then, in Los Angeles, the use of magnesium anodes has proved to be by far the most economical system for cathodic protection of water tanks and has resulted in substantial savings for the Los Angeles water system.

In experience with the waters described in Table 1, the magnesium anode offers the following important advantages:

1. It is self-energizing and trouble-free. The external power supply (rectifier), accessory a-c feeder lines, and other equipment normal to the graphite anode installation are eliminated, along with the attendant installation and maintenance costs.

2. The voltage developed by magnesium is ample for the protection of tanks coated with coal-tar enamel, and yet is low enough to avoid any discernible damage to such coatings. This probably results from the self-regulatory feature of magnesium anode installations, whereby the current output of the anode tends to diminish as potentials at the tank wall approach values capable of causing damage to the coating. Even in certain bare-steel tanks, where the quantity of magnesium needed might appear to be excessive, magnesium anodes have proved to be effective and practical sources of protective current if waters of high mineral content, like some described in Table 1, are involved. This probably results from the decrease in current requirements attending the deposition of calcium carbonate scales and would not be realized with poorly mineralized waters containing only a small amount of calcium ion.

3. The magnesium anode offers a high degree of flexibility and versatility. The magnesium rods employed can be readily formed, grouped, or otherwise positioned so as to accommodate unusual tank geometry and concentrate the protective current at those areas where it is most needed, as was shown in Lakeside tank when magnesium anodes were installed. This is a valuable feature, especially when it becomes necessary to throw current into

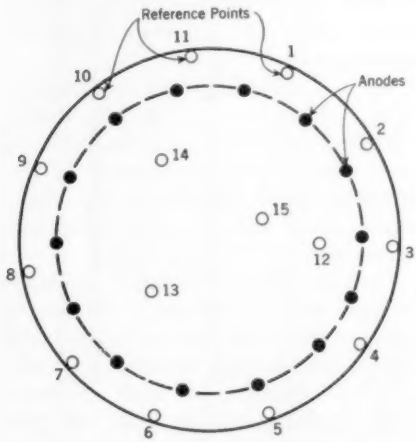

Fig. 2. Locations of Anodes and Test Holes in Topanga Tank

The fourteen magnesium anodes used in the tank were 0.84 in. in diameter and 25 ft long.

sharp corners and other semirecessed portions of the tank structure.

Topanga Tank

The water contained in Topanga tank has approximately the analysis shown in Column *A* of Table 1. This tank is 36 ft in diameter by 30 ft high, with an operating capacity of 208,000 gal. It is a steel tank located on the northerly slope of a hill, its bottom resting directly on a pad of oiled sand laid on a leveled area cut out of the hillside. At the time cathodic protection was installed, the enamel had peeled off the inside of the tank in several small spots, corresponding to locations exposed to direct sunlight on the outer surface of the tank. Some rusting and corrosion had started in these areas.

Fourteen 0.84-in. diameter magnesium anodes 25 ft in length, were installed in March 1948, without the usual cathodic-protection requirement survey. Figure 2 shows the position of the magnesium anodes and test holes or reference points (RP) in this tank.

After a full year of cathodic protection, the tank was recoated with coal-

TABLE 3

Weights and Weight Losses in Anodes After 1 Year

Anode No.	Original Weight—*lb*	Final Weight—*lb*	Loss of Weight—*lb*
1	11.15	10.585	0.565
2	11.15	11.055	0.095
14	11.15	10.995	0.155
Total	33.45	32.635	0.815

tar enamel of the best quality. At this time, the three anodes located nearest the previously noted areas of bare metal showed some evidences of weight loss. They were therefore removed, dried, cleaned, weighed and reinstalled. The respective weights and weight losses of these anodes are shown in Table 3. These data show annual weight losses ranging from 0.1 to 0.6 lb per anode for those anodes subjected to the highest current drains—that is, those anodes nearest uncoated metal. For the anode weights employed (11.15 lb), this represents an estimated minimum anode life of about 18 years, assuming complete utilization of anode metal. Although the latter assumption is mod-

estly optimistic, 18 years is considered to be highly satisfactory performance inasmuch as complete protection was afforded to a tank whose coating was somewhat defective before cathodic protection was applied. As will be shown later, the total current output of all anodes after the second application of coal-tar enamel was a mere 0.027 amp. This probably indicates a further reduction in the rate of magnesium consumption. Although these data are evidence of outstandingly economical protection, they are typical of the performance of magnesium anodes under the conditions prevailing in the Los Angeles system.

Topanga tank has now been under continuous cathodic protection since March 1948. Inspections during this time have shown no observable deterioration of either the tank metal or the protective enamel coating. Only the float mechanism, which is not connected to the cathodic-protection system, shows any evidence of corrosion.

Tests made in February 1953 on Topanga tank gave the following results: depth of water above tank bottom, 27 ft; cathodically protected area (wall and bottom), 4,070 sq ft; average temperature of contained water, 56°F; total current drawn, 0.027 amps; water resistivity, 4,500 ohms per cubic centimeter; tank-to-water potentials at RP No. 1 (1 ft from tank bottom), 1.22 v; and tank-to-water potential at RP No. 10 (24 ft from tank bottom), 1.3 v. A potential profile for the reference points at Topanga tank is given in Fig. 3.

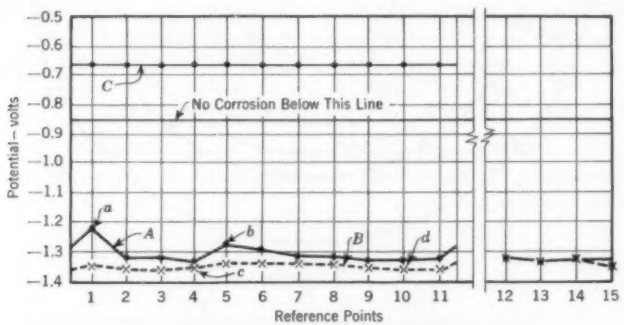

Fig. 3. Potential Profiles for Topanga Tank

Lines A *and* B *indicate potentials with cathodic current connected. Tank-to-water potentials at least-protected positions on the reference points are indicated by Line* A. *Tank-to-water potentials at most-protected positions are indicated by Line* B. *Line* C *represents potentials before anodes were connected. Heights above tank bottom for a few significant potentials are:* a—*1 ft;* b—*26 ft;* c—*24 ft; and* d—*24 ft.*

The extremely low values of current flow needed to give this tank complete protection indicate the excellence of the coal-tar enamel coating. It also points to the fact that the magnesium anode installation, consisting of 14 anodes, is greatly overdesigned for the present condition of the tank coating, although it may have been appropriate at the time the installation was made and before recoating.

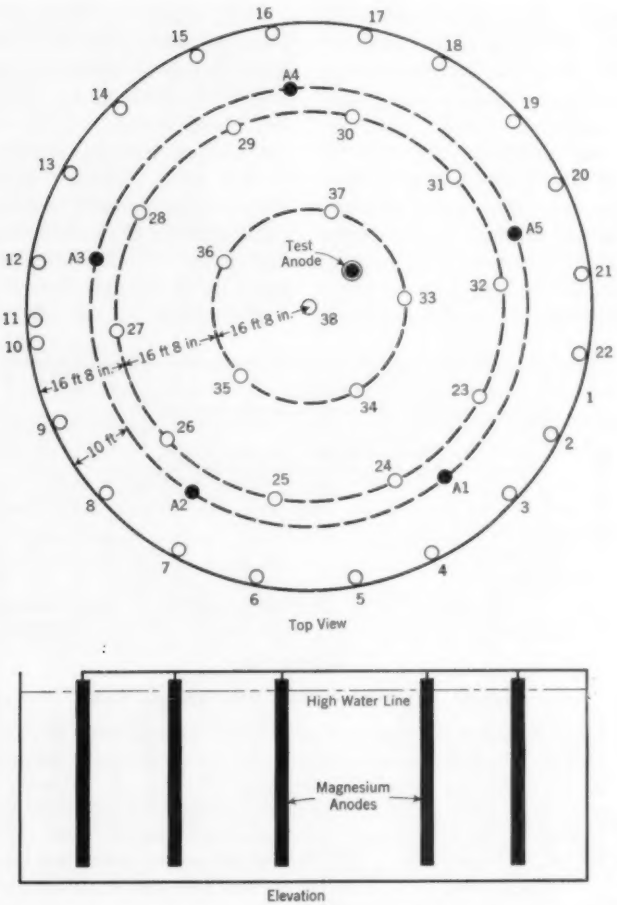

Fig. 4. Locations of Anodes and Test Holes in Coldwater Canyon Tank

The black circles, A1-A5, are magnesium anodes. The test anode for the cathodic-protection survey, shown near the center of the tank, was also of magnesium. A large automobile storage battery was connected between this test anode and the tank wall.

Coldwater Canyon Tank

The cathodic-protection installation used on the Coldwater Canyon tank is a good example of the recommended method of installation now adopted as standard practice by the Los Angeles water department.

Water contained in Coldwater Canyon tank is generally taken from sources yielding water of the analysis

given under Column *A* of Table 1, with admixtures of smaller quantities of water from other sources. Its electrical resistivity was 7,000 ohm per cubic centimeter.

The Coldwater Canyon tank has a diameter of 100 ft and a height of 40.5 ft. Its capacity is 2,255,000 gal. At time of installation, the protective coating was in excellent condition above the water line and appeared good below the water line. Locations of test holes are shown in Fig. 4.

The first step was the installation of test holes in the tank roof through which the copper sulfate electrode could be lowered to measure the potentials of the tank walls and bottom before and during application of protective current. Test holes were so distributed as to permit a complete survey of the tank. A magnesium test anode was then placed near the center of the tank. Data are given in Tables 4 and 5.

A large automobile storage battery was connected between the test anode and a metallic connection to the tank wall through a variable-resistance and a low-resistance ammeter, as shown in Fig. 5. The total cathodic-protection test current applied was adjusted with the variable resistance until the tank-to-water potential at the least protected point was more than 0.85 electronegative, as measured with a copper sulfate electrode. The copper sulfate electrode, which was connected to the tank wall through a sensitive high-resistance voltmeter, was placed at various locations close to, but not in contact with, the tank wall. Exact locations of these voltage readings were recorded by reference points, or test holes, as shown in Table 5 and plotted in Fig. 6.

Reference points showing the least potential difference between water and tank were considered to be the least protected. Such points required the greatest concentration of cathodic-protection current under operating conditions. Likewise, reference points showing the greatest difference of potential were considered to be the most protected points, and required least concentration of current.

With 0.475 amp flowing from the test anode to the tank, a tank-to-water potential of −1.03 was measured with the copper sulfate electrode at the least-protected reference point, RP No. 11, 3 ft above tank bottom, and a tank-to-water potential of −1.25 v was measured at the most-protected reference point, RP No. 21, at 28 ft above

TABLE 4

Cathodic-Protection Survey—Coldwater Canyon Tank *

Test Location		Tank-to-Water Potentials†—v	
Reference Point No.	Height Above Tank Bottom—*ft*	Anodes Disconnected	With Cathodic-Protection Current of 0.475 amp
5	34	−0.52	−1.07
11	3	−0.54	−1.03
18	30	−0.52	−1.12
21	28	−0.53	−1.25
22	0	−0.54	−1.08

* The survey was made on Dec. 18, 1952.
† Current output of one test anode, without battery, was 0.145 amp.

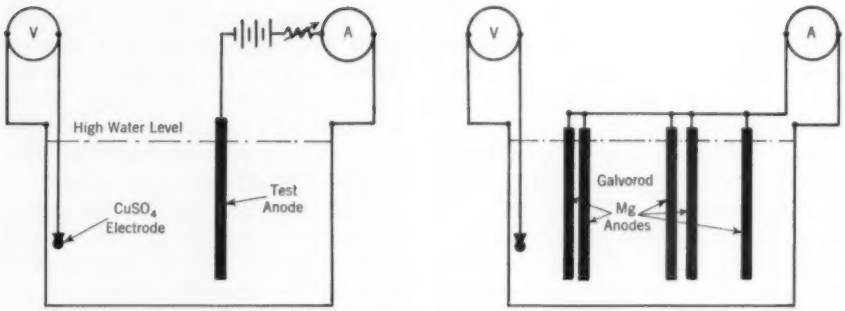

Fig. 5. Electric Circuits for Coldwater Canyon Tank

The circuit diagrammed at the left was for the cathodic-protection survey. That at right was for testing the adequacy of cathodic protection.

tank bottom. Both of these voltages indicated a tank-to-water potential sufficiently electronegative to suppress the anodic dissolution of ferrous ions, and the tank was therefore considered to be fully protected with 0.475 amp. The lower and upper limits of −0.85 and −1.40 v have been tentatively established for operation of the Los Angeles cathodic-protection systems in water tanks.

The current from anode to tank, without any external current supply from the battery, was 0.145 amp. The total cathodic protection–current requirement of 0.475 amp divided by the current output of 0.145 amp from one anode gave 3.3 as the number of anodes necessary in this tank for adequate protection under conditions prevailing at the time of test. Four such anodes, each 1.3 in. in diameter by 37.5 ft long, appeared to be the correct number to install. But since most protective coatings are expected to deteriorate further with age, sufficient anodes should be permanently installed to give somewhat more than the indicated current requirement of the tank at time of test. Allowing a 25 per cent safety factor, five such anodes were permanently installed in the Coldwater Canyon tank and arranged as in Fig. 4.

TABLE 5

*Tank-to-Water Potentials With Five Magnesium Anodes and No External Source of Cathodic Protection**

Test Location		Tank-to-Water Potentials—v	
Reference Point No.	Height Above Tank Bottom—*ft*	Anodes Disconnected	With Cathodic-Protection Current of 0.43 amp
5	34	−0.53	−1.07
11	3	−0.54	−1.03
18	30	−0.54	−1.11
21	28	−0.54	−1.10
22	0	−0.55	−1.12

* These tests were made on Jan. 9, 1953. Before connecting anode installation to tank, time was allowed for polarization potentials and films established during cathodic-protection requirement survey to become dissipated.

General Practice

A general rule which is convenient and practical for the spacing of anodes is to space them around a concentric circle within the tank so that the radius of the circle is 0.354 times the diameter of the tank. This preliminary plan is then modified when installing the anodes to correct substantially the potential profile as shown in Fig. 6.

life of magnesium anodes at Los Angeles have actually been exceeded, if the present condition of anodes now in service for several years is a criterion by which to judge.

After the anodes are installed, initial formation of polarizing films of hydrogen and their subsequent gradual displacement by tightly adhering calcium carbonate scale can be more quickly

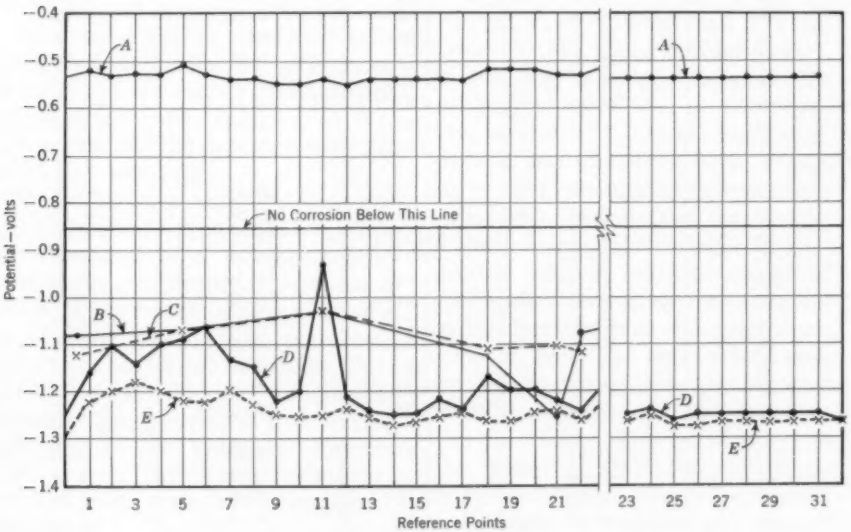

Fig. 6. Potential Profiles for Coldwater Canyon Tank

Dates of tests from which readings were obtained are: Line A*—Dec. 15, 1952, before cathode protection; Line* B*—Dec. 18, 1952; Line* C*—Jan. 9, 1953; Line* D*—Mar. 12, 1953; and Line* E*—Mar. 12, 1953. Line* D *indicates positions on reference points which were least protected and Line* E*, positions which were most protected, according to tests made on Mar.12, 1953.*

Nomographs are available from manufacturers of magnesium anodes for determining current output, life, weight, or efficiency of the anodes. With these values and the cathodic-protection requirements from the tests, an estimate of the life expected from a magnesium anode installation can be made. Earlier estimates of the

established by temporarily connecting an automobile battery in series with the anodes. Such operation, for 1 or 2 days only, will usually accomplish the purpose, after which the battery may be removed and the magnesium anodes connected directly to the tank.

At this point, it is necessary to realize that, whenever any external

TABLE 6

Tank-to-Water Potentials at Least-protected and Most-protected Locations of Coldwater Canyon Tank *

Reference Point No.	Least Protected		Most Protected	
	Height Above Tank Floor—*ft*	TW Potential—*v*	Height Above Tank Floor—*ft*	TW Potential—*v*
1	34–36	−1.16	13–24	−1.22
2	0– 1	−1.10	9–30	−1.20
3	34–36	−1.14	1–27	−1.18
4	34–36	−1.10	8–18	−1.19
5	35–36	−1.09	0–16	−1.22
6	35–36	−1.06	0–19	−1.22
7	34–36	−1.13	0–24	−1.20
8	35–36	−1.14	0–21	−1.22
9	34–36	−1.22	14–28	−1.25
10	0	−1.20	15–36	−1.25
11	1	−0.93	15–36	−1.25
12	0– 6	−1.21	10–36	−1.24
13	0–14	−1.24	15–36	−1.25
14	0– 2	−1.25	14–36	−1.27
15	0– 7	−1.25	8–36	−1.26
16	0– 2	−1.22	14–36	−1.25
17	0–36	−1.24	0–36	−1.24
18	32–36	−1.17	3–22	−1.26
19	30–36	−1.20	0–20	−1.26
20	29–36	−1.20	0–22	−1.24
21	29–36	−1.22	0–25	−1.24
22	0– 4	−1.24	17–26	−1.26
23	0–17	−1.25	18–36	−1.26
24	0–36	−1.24	0–36	−1.21
25	0–36	−1.26	0–36	−1.26
26	0–36	−1.26	0–36	−1.26
27	0–13	−1.26	26–36	−1.28
28	0– 2	−1.25	15–36	−1.27
29	0– 3	−1.25	16–36	−1.27
30	0– 8	−1.25	9–36	−1.26
31	0–36	−1.26	0–36	−1.26
32	0–15	−1.25	16–27	−1.26
33	0– 6	−1.24	7–36	−1.25
34	0–14	−1.25	15–36	−1.26
35	0– 5	−1.25	6–36	−1.26
36	0– 3	−1.25	4–36	−1.26
37	0–36	−1.26	0–36	−1.26
38	0–10	−1.25	11–36	−1.26
39	0	−1.26	0	−1.26

* At depths not shown, tank-to-water potentials varied between limits given.

source of current is applied, such as by series connection of a storage battery or rectifier in the circuit, sufficient electrical resistance must be inserted to limit the tank-to-water potentials across protective enamel coatings so that voltages across the coating will not exceed critical values, above which deterioration of the coating may occur. The voltages actually applied may be measured by the method illustrated in the right-hand drawing in Fig. 5.

After the polarizing films have been established, new potential surveys can be made and the anodes adjusted until the potential profile, as shown in Fig. 6, is well within the protected potential area at all locations.

The experience with the Topanga and Coldwater Canyon tanks illustrates the advantages of a current requirement survey in advance of installation design if maximum economy in the use of anode material is to be realized. The extent of this saving may be illustrated by comparing the installation in Topanga tank, which was unsurveyed, with that of the Coldwater Canyon tank, where a current requirement survey was made prior to the permanent installation.

Topanga tank, which has a capacity of approximately 208,000 gal, is being protected with 350 ft of 0.84-in. magnesium rod, whereas Coldwater Canyon, which has a 2,255,000-gal capacity, is adequately protected with only 187.5 ft of 1.3-in. magnesium rod. The difference in rod diameter is not significant as regards current output per unit length of rod. The above difference in installation sizes might be accounted for on the basis of differences in coating condition or water conductivity and corrosiveness, except that operating data show a lower current density requirement for the Topanga tank, which is being protected with a current density of some 6 ma per 1,000 sq ft of tank surface. The Coldwater Canyon tank requires approximately 21 ma per 1,000 sq ft to insure effective protection.

Conclusions

After more than 10 years of practical experience with cathodic protection of water tanks, and after trying several different anode materials and different systems for energizing the installations, the Los Angeles water system has now standardized upon the use of magnesium anodes as the most economical, effective, and versatile system for protecting its tanks against corrosion.

Best economy has been obtained by the use of magnesium anodes in conjunction with a coal-tar enamel of good quality, but adequate protection has also been observed with magnesium anodes in bare-steel tanks where tank size and water conditions—that is, conductivity and calcium ion content—were favorable. The latter result is most probable where the dissolved-mineral and carbon (Ca^{++}) ion content of the water is high and the current requirement is low, as in small tanks.

The experimentation has resulted in the development of designs and methods of installation and testing, whereby the corrosion engineer should be able very nearly to obtain the exact cathodic-protection requirements for any water tank.

The designs and installation practices described here are adequate for tanks operating under mild climatic conditions such as those found in Southern California. Considerable modification may be required for good serviceability where winters are cold and severe icing is encountered.

Some Novel Approaches to Population Studies

K. L. Kollar

A contribution to the Journal by K. L. Kollar, San. Engr., Inst. of Inter-American Affairs, Caracas, Venezuela.

ALMOST all existing methods of population prediction involve a human factor in the processes of rounding off and extension. During an unpublished discussion in 1953, Abel Wolman remarked the absence of any major advance in extending the scope of existing methods of population prediction. Observing that virtually the only efforts in this direction are attributed to economists and demographers, Wolman described a promising new method advanced by the Australian economist Colin Clark. These remarks stimulated the author, a student of Wolman, to review Clark's contributions in more detail.

In the paper "Urban Population Densities," published in 1951 (*1*), Clark, like Wolman, comments upon the lack of study in this field. Of the few existing works on population densities, he praises those of Mueriot and Mark Jefferson. Among more recent studies, he speaks highly of the work of Griffith Taylor (*2*).

The work involved in the collection and correlation of data for the preparation and detailed study of several density maps is enormous. To simplify this task, two hypotheses have been generally accepted:

1. In every large city (not including the central business section, which has few residents), population density is greatest in the interior and declines progressively toward the outer suburbs.

2. In most cities, a gradual outward dispersion occurs: densities decrease in the most populous inner suburbs, and increase in the outer suburbs.

On the basis of these hypotheses and population data, Clark proposes the following equation:

$$y = Ae^{-bx}$$

in which y is the density (in thousands) of resident population per square mile; x is the distance (in miles) from the center of the city; b is the coefficient of the rate of decline of density; A is the coefficient of the density at the center of the city; and e is the natural logarithmic base.

Following this equation, then, the total population within a radius r is:

$$\int_0^r Ae^{-bx}(2\pi x)\,dx$$

$$= 2A\pi \int_0^r e^{-bx}x\,dx$$

$$= 2A\pi \left[\frac{e^{-bx}}{b^2}\left(\frac{1}{bx}\right)\right]_0^r$$

$$= 2A\pi \left[1 - e^{-br}\left(\frac{1}{br}\right)\right]$$

when r is ∞ (infinity). Thus, to obtain the total population of the city (in thousands) this becomes:

$$\frac{2A\pi}{b^2}$$

TABLE 1

Values for the Equation $y = Ae^{-bx}$

City	Population—1,000's	*A*	*b*	*b*/Pop.
New York				
1900	3,437	250	0.55	0.000062
1910	4,767		0.20	0.000042
1940	7,450	120	0.20	0.000027
Chicago				
1900	1,699	110	0.45	0.000265
1940	3,397	120	0.30	0.000885
Boston				
1900	509	160	0.85	0.00168
1940	771	50	0.30	0.00386
Cleveland				
1940	878	90	0.45	0.000512
Los Angeles				
1940	1,504	30	0.25	0.000166
Philadelphia				
1900	1,294	120	0.65	0.000502
1940	1,931	60	0.40	0.000207
St. Louis				
1900	575	70	0.75	0.00131
1940	816	40	0.45	0.00055
London				
1801	959	290	1.35	0.00141
1841	2,235	800	1.40	0.000630
1871	3,260	290	0.65	0.000199
1901	4,537	210	0.45	0.000099
1921	7,480	180	0.35	0.0000467
1939	8,700	80	0.20	0.000023
Paris				
1817*	650	450	2.35	0.00361
1856*	1,300	240	0.95	0.00073
1896*	2,540	370	0.80	0.000315
1931*	2,820	470	0.75	0.000265
Berlin				
1885	1,315	290	1.10	0.000835
1901*	1,520	410	0.95	0.000625
Liverpool				
1921	900	330	0.80	0.00089
Melbourne				
1925	1,000	100	0.55	0.00055
Manchester				
1931	766	40	0.25	0.000316
Vienna				
1890	900	170	0.80	0.00089

* Records incomplete.

Values of A and b for various cities are shown in Table 1.

Theoretical Basis

Since no further explanation of the derivation of this equation is given by Clark, a brief discussion is included here. A possible basis for this empirical formula, and an interesting explanation of the reasoning involved, have been expounded by the astronomer James Q. Stewart, who refers to his analogy as "social physics" (*3, 4*). Stewart recognizes the foresight of the famous physicists Boyle, Charles, and Gay-Lussac in taking the macroscopic view of gases and investigating the quantitative fields of density, temperature, and pressure, instead of taking the microscopic view and awaiting the results of further investigations of molecular activity. With regard to population, however, Stewart suggests that the microscopic view is applicable: that is, that people may be studied in terms of their relations as individuals or "molecules." Acting on this assumption, he proceeds to apply Newton's law of universal gravitation—namely, that each particle of matter attracts every other particle with a force directly proportional to the product of their masses, and inversely proportional to the square of the distance between them. Stated otherwise:

$$F = \frac{Gm_1m_2}{d^2}$$

in which F is the force of attraction, G gravitational force, m_1 and m_2 the particular masses, and d the distance

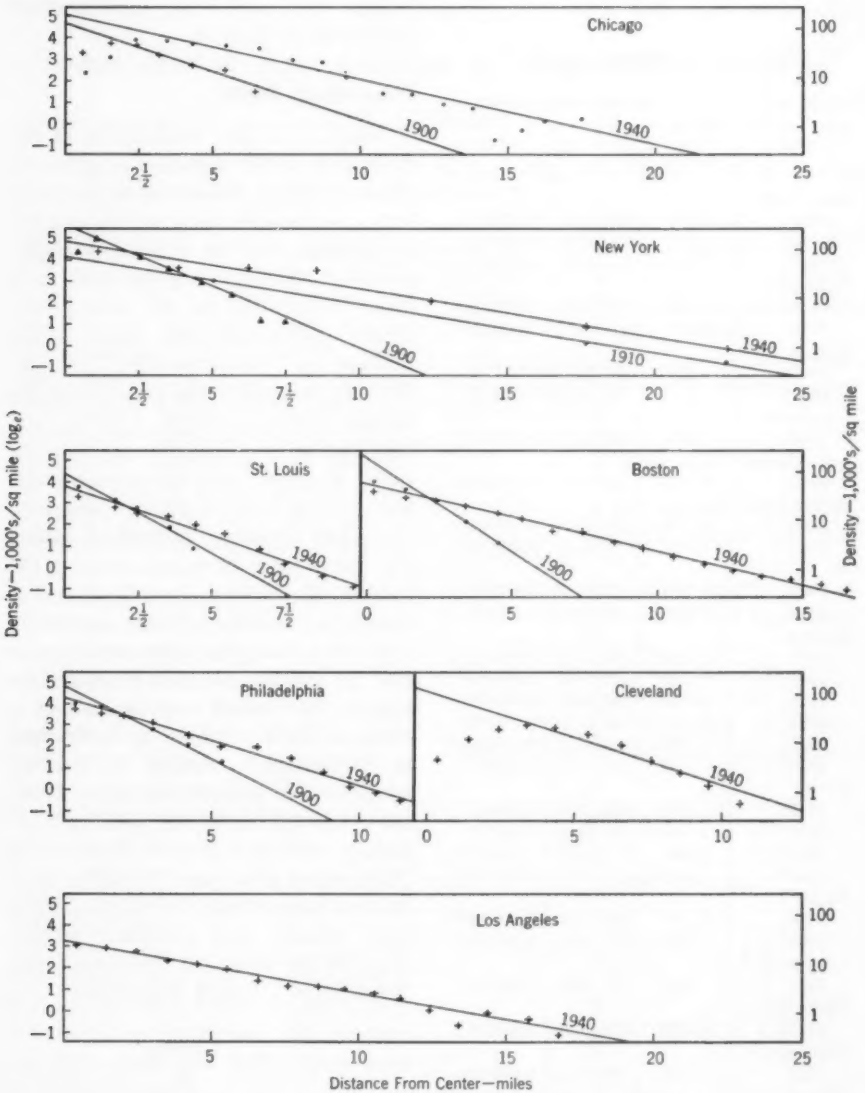

Fig. 1. Clark's Equation Plotted for American Cities

The vertical axes are calibrated on the left in natural logarithms of the density (in thousands per sq mile), and on the right in thousands per sq mile. The horizontal axes indicate distance from the center of the city (in miles).

between them. Then the mutual energy of the two masses in the gravitational field may be stated:

$$E = \frac{Gm_1m_2}{d}$$

and the forces or potentials, V_A and V_B, produced by each mass at points A and B, are expressed by the equations:

$$V_A = \frac{Gm_1}{d}$$

$$V_B = \frac{Gm_2}{d}$$

On the basis of these formulas, Stewart derives:

$$2E = m_2V_A + m_1V_B$$

When, instead of two particles, we are dealing with a number of particles, the total potential is the sum of the separate potentials; and if the distribution of mass is confined to a plane surface, and is continuous, the potential at any point C may be stated:

$$V_C = \frac{D}{r}(ds)$$

in which D is the surface density of the mass over the infinitesimal area ds, and r is the distance from that area to the point C. (The integration is extended to all areas of the plane in which D is zero.) From this, a contour map of equipotentials may be drawn. Stewart, of course, relates mass to people, area to the area of a city, and r to the distance from the center of the city, to apply this formula to economic studies. After this elaborate derivation, however, he proposes in closing, an empirical formula:

$$A_C = \frac{P^{3/4}}{400}$$

which for the 1940 census was changed to:

$$A_C = \frac{P^{3/4}}{357}$$

in both of which A_C is area, and P is population.

Application of Clark's Equation

Returning to Clark's formula, the relation stated $y = Ae^{-bx}$ appears to be true for the period from 1801 to the present time, and for cities ranging from Los Angeles to Budapest. The proof of the formula has been carried out by calculating the total population, and the average densities of a series of concentric rings (generally drawn at each mile of radius) about the center of the city. The determination of the coefficients A and b was comparatively simple: they were computed by simultaneous equations, and the values for each ring averaged. Graphs were then plotted (Fig. 1 and 2), in which the horizontal axis represents the distance (in miles) from the center of the city, and the vertical axis represents the natural logarithm of the density (in thousands per square mile) as derived from the equation:

$$\log_e y = \log_e A - bx$$

On these graphs, all data are plotted as straight lines, with the exception of those relating to the business districts.

As seen in Fig. 1 and 2, A is the point at which the line cuts the vertical axis, and b is the slope. In other words, the value of b controls the rate of decline of the density: a high value

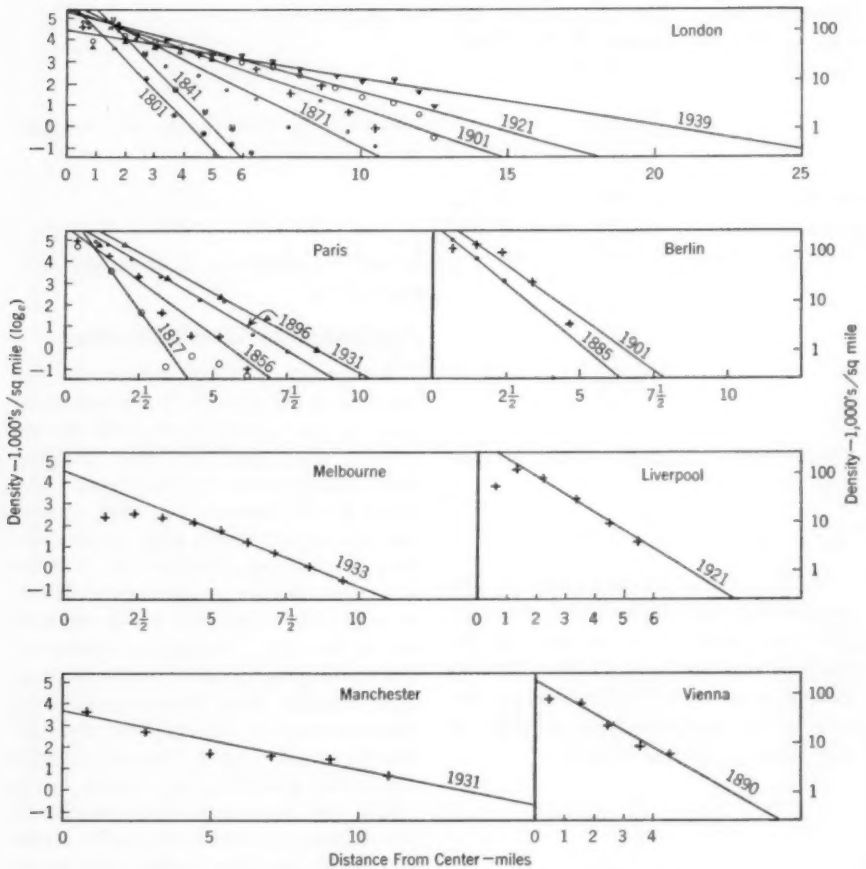

Fig. 2. Clark's Equation Plotted for European Cities

The points at which the lines cut the vertical axes are values of A, *the coefficient of the density at the center of the city. The slope is* b, *the coefficient of the rate of decline of density. Thus, the greater the slope, the more sharply density decreases from the center of the city outward.*

of b means the density will decrease sharply from the center outward, a low value that the density will decrease slowly, indicating a spreading-out effect. Clark attributes the slope to the relationship between costs of transportation and average income. Spreading out, he contends, occurs only when transportation costs are low. In this respect, the coefficient A may be regarded as an indication of the degree of overcrowding that people are prepared to tolerate at the center of the city. Thus, in the hypothetical circumstance

in which the value of X is zero, y approaches A. (There is not a complete equality because of the low densities at the center of the city.)

Examining the data in Fig. 1 and 2, it may be observed that in the case of London, New York, Chicago, and Los Angeles, the value of b is approaching or equal to 0.2. In New York and London, Clark attributes this to low-fare subway systems; in Chicago, to the surface railway; and in Los Angeles, to the high rate of automobiles per capita.

Similar values of b (indicated by parallel lines in Fig. 1 and 2) suggest similar patterns of dispersal and growth. Those of Manchester and Los Angeles, for example, are similar —although, of course, for different eras. Los Angeles' dispersal is usually attributed to the fact that its growth occurred almost entirely in the automobile age; while Manchester's growth, in the 19th century, was affected in a similar manner by the presence of tram cars and steam railways. Factories and other places of employment are well distributed throughout Manchester and environs; and Los Angeles, in Clark's opinion, is becoming quite similar to the English city.

A gross decline in the value of b for a single city, indicating greater outward dispersion, may be traced to a significant decrease in the cost of transportation. The decline in New York City, for example, from 1900 to 1910, is attributed to the introduction of subways and other low-fare transportation facilities in that period. During the period 1910–1940, the absence of a major reduction in transportation costs caused a comparatively uniform increase of density in all directions, as shown by the parallel lines on the graph. Similarly, the change in London, from 1841 to 1871, coincides with an increase in horse-drawn and steam transportation in that city. In Paris, from 1856 to 1896, electrification of the transportation system took place: outward dispersion, however, was minimized by the high fares (in a period of generally low wages) and the traditionally long noon meal hour (which allowed workers to return home at noon and thus served to limit distances between residences and places of employment). Electrification produced a more significant change in Boston between 1900 and 1940.

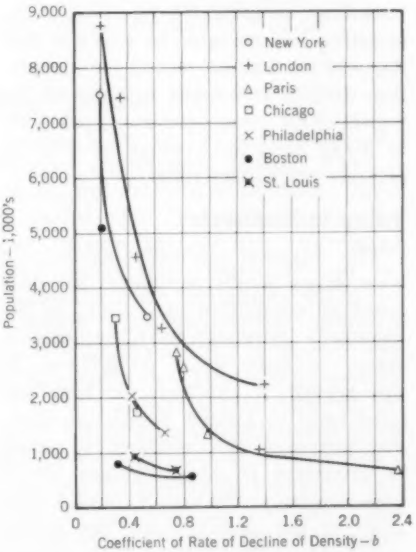

Fig. 3. Population Growth in Relation to Density Decline

Horizontal coordinates were determined by Clark's equation, in which they are values of b. *For purposes of population prediction, more complete data relating to longer periods are necessary.*

Of all cities shown for 1900, Chicago had the lowest value of b: this is in agreement with records of the period which indicate that, by the standards of the time, Chicago was considered to have a high degree of dispersal, and that the inhabitants enjoyed relatively good health. For 1901, London had the lowest b, but suffered severe overcrowding, as indicated by A. An interesting fact revealed by Clark is that the world's known record for population density was made in parts of the Lower East Side of New York City in 1900, with an average of 350,000 persons per square mile.

Value to Engineers

The question naturally occurs: Of what value is this technique, for purposes of population prediction, to the engineer? Although the Clark formula seems to have good possibilities in this regard, it has thus far been applied only to known population data. For purposes of prediction, it would be necessary to obtain values for A (the density at the center) and b (the slope of the density from the center outward). The author has plotted the available values against population statistics for the corresponding periods, as shown in Fig. 3. On the basis of this graph, it appears that each city has a unique curve. It would be wise, however, to withhold any further conclusions until the method has been applied to smaller cities and (where possible) over longer periods for which more complete records exist. The data available in 10-year censuses for United States cities, from 1900 to the present, would serve this purpose admirably. With such information it would be possible to determine how the missing values may be calculated for any city.

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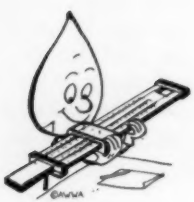

Theory of Sedimentation

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LIKE many other natural phenomena utilized in water treatment, sedimentation was put to practical use in the clarification and purification of water long before serious attempts were made to explore the complex laws which govern its behavior. During the past half century, however, a vast amount of theoretical and experimental consideration has been given the subject. That the results have been vexingly modest in terms of criteria for the engineering design of effective and economical sedimentation basins bespeaks the extreme complexity of the problem rather than any lack of inspiration or diligence on the part of its investigators. During the past 10 years, important strides have been made in simplifying the development of the laws of sedimentation from empirical data and in solving the resulting equations, as well as in isolating and evaluating the effects of individual variables. Nevertheless, no satisfactory or workable procedure has been developed for translating theory into practice. Sedimentation tank design necessarily continues to be based on current practice, modified by the judgment of the designer as derived both from experience and from application of the laws of sedimentation.

Basic Theory

The laws of sedimentation developed by Stokes, Hazen, Newton (*1–3*), and other investigators appear in many texts and reference books. For various reasons the authors of such books have generally not included the basic derivations which the newcomer to sedimentation problems must understand if he is to use them intelligently as a guide to judgment. It is the purpose of this paper to set forth the origins and relationships of the laws of sedimentation commonly reported in engineering literature as a result of experimental and theoretical considerations, and to review some of the problems involved in adapting them to the design of practical settling tanks.

Basically, the theory of sedimentation is the theory of the effect of gravity on particles suspended in a liquid of lesser density. Under the influence of gravity, any particle having a density greater than that of water will settle in water at an accelerating velocity until the resistance of the liquid equals the effective weight of the particle. Thereafter, the settling velocity will be essentially constant and will depend upon the size, shape, and density of the particle, and the density and viscosity of the

water. For most theoretical and practical computations of settling velocities in sedimentation basins, the shape of the particles is assumed to be spherical. Settling velocities of particles of other shapes can be analyzed in relation to spheres. For a spherical particle of diameter d and density ρs, in a liquid of density ρ, the accelerating force F equals the weight of the particle minus its buoyancy force. When the resisting force R equals the accelerating force, a constant settling velocity is achieved. This is expressed by:

$$R = F = \frac{\pi d^3}{6} g(\rho s - \rho) \ldots . (1)$$

The resistance of a liquid to the passage through it of a solid sphere is a function f of four independent variables: the viscosity μ and density ρ of the liquid, and the diameter d and velocity v of the object. This may be expressed by the equation:

$$R = f(d, v, \rho, \mu)$$

This equation may be studied by dimensional analysis by putting it in the form:

$$R = K(d^w v^x \rho^y \mu^z) \ldots . (2)$$

Substituting the mass, time, and length units (m, t, l) involved in the definitions of force, diameter, velocity, density, and viscosity, this equation becomes:

$$[mlt^{-2}] = K[l^w (lt^{-1})^x (ml^{-3})^y (ml^{-1}t^{-1})^z]$$

Because exponents on each side must balance, for mass, $m = m^y m^z$ or $1 = y + z$, and $y = 1 - z$. For time, $t^{-2} = t^{-x} t^{-z}$, or $-2 = -x - z$, and $x = 2 - z$. For length, $l = l^w l^z l^{-3y} l^{-z}$, or $1 = w + x - 3y - z$, and $w = 2 - z$.

Substituting in Eq 2:

$$R = K[d^{2-z} v^{2-z} \rho (dv\rho)^{-z} \mu^z]$$

$$= K d^2 v^2 \rho \left(\frac{\mu}{dv\rho}\right)^z$$

This may be written as:

$$R = (k_1 d^2 v^2 \rho) k_2 \left(\frac{dv\rho}{\mu}\right)^{-z}$$

in which $k_1 \times k_2 = K$; or more simply $\left(k_1 \frac{R}{d^2 v^2 \rho}\right)$ = a function of $\left(\frac{dv\rho}{\mu}\right)$, a dimensionless quantity known as Reynolds Number (**R**). The term $\left(\frac{k_1 R}{d^2 v^2 \rho}\right)$ is called the "coefficient of drag" (C_D), and:

$$C_D = k_1 \frac{R}{d^2 v^2 \rho} = f\mathbf{R} \ldots . (3)$$

Combining Eq 1 and 3:

$$C_D = k_1 \frac{\frac{1}{6}\pi d^3 g(\rho s - \rho)}{d^2 v^2 \rho} = f\mathbf{R}$$

or:

$$C_D = k_1 \frac{\pi}{6} \frac{d}{v^2} g \left(\frac{\rho s - \rho}{\rho}\right) \ldots (4)$$

and:

$$C_D = k_2 \left(\frac{dv\rho}{\mu}\right)^{-z} = f\mathbf{R} \ldots (5)$$

The term (k_1) can be evaluated by comparing Eq 3 with an equation presented by Newton (*3*), and now usually expressed as:

$$R = C_D \frac{A\rho v^2}{2} \ldots . (6)$$

in which A is the projected area of particle in direction of motion $\left(\frac{\pi d^2}{4}\right.$ for spheres$\left.\right)$ and other terms are as previously defined.

Equation 6 now becomes:

$$C_D = \frac{8}{\pi} \frac{R}{v^2 d^2 \rho}$$

From this and Eq 3 it follows that $k_1 = \frac{8}{\pi}$, and Eq 4 therefore becomes:

$$C_D = \frac{8}{\pi} \frac{\pi}{6} \frac{d}{v^2} g \left(\frac{\rho s - \rho}{\rho} \right)$$

$$= \frac{4}{3} \frac{d}{v^2} g \left(\frac{\rho s - \rho}{\rho} \right)$$

Solving for v, the general equation for the settling velocity of a sphere, commonly known as Newton's Law, is developed:

$$v = \sqrt{\frac{4}{3} \frac{d}{C_D} g \left(\frac{\rho s - \rho}{\rho} \right)} \quad \ldots (7)$$

Although particles settling through a liquid are resisted by forces of both inertia and viscosity, Newton originally considered only inertia. He therefore assumed that C_D was a constant and that the frictional resistance varied as the square of the velocity. By analyzing the equation $R = f(d, v, \rho, \mu, g)$ dimensionally, Vennard (*4*) has shown rationally that C_D in Newton's equation (Eq 6) is a function of both Reynolds number (**R**) and Froudes number. Experiments have shown many times that C_D varies with the Reynolds number in a manner such as shown in Fig. 1, being essentially constant only in the range of **R** values of 1,000–250,000.

Camp (*5*), Straub (*6*), and others have shown that particles, of such diameter that viscous forces predominate, settle under streamline conditions, resisted by a force which is proportional to the first power of the velocity. Particles of larger sizes, however, are influenced by both viscosity and inertia, and they encounter resistances proportional to some power of the velocity greater than one. When viscous forces become small in comparison with inertia forces, the resistance is determined entirely by inertia and the resisting force (R) varies with the square of the velocity. Thus, it is evident that Eq 7, with a constant value of C_D, is limited in its application and represents the well established experimental curve in Fig. 1 only in the higher range of **R**, where flow is turbulent.

Equation 7, as well as equations which define the laws of sedimentation governing the subsidence of particles at the lower values of **R**, where flow is streamline or transitional, may be developed from Eq 5. On the basis of the observed fact that the power of v varies from one to two under various flow conditions, the expression may be put in the form:

$$C_D = k\mathbf{R}^{n-2}$$

in which n represents the state of flow—that is, $n = 1$ for streamline flow, n is more than 1 but less than 2 for transitional flow, and n is 2 for turbulent flow. Then:

$$C_D = \frac{8}{\pi} \frac{\pi}{6} \frac{d}{v^2} g \left(\frac{\rho s - \rho}{\rho} \right)$$

$$= k \left(\frac{vd\rho}{\mu} \right)^{n-2} \quad \ldots (8)$$

Simplifying and solving for v^n, the equation becomes:

$$v^n = \frac{4}{3} \frac{g}{k} \left(\frac{\rho s - \rho}{\rho^{n-1}} \right) \mu^{n-2} d^{3-n} \quad \ldots (9)$$

which is the general equation for the settling velocity of particles under any flow conditions.

Stokes' Law

As previously noted, when the particle diameter d is sufficiently small the predominance of viscous forces results in accelerative and resistive forces proportional to the first power of the velocity. That is, $n = 1$ in Eq 7. Equation 7 then becomes:

$$v = \frac{4}{3}\frac{g}{k}\left(\frac{\rho s - \rho}{\mu}\right) d^2 \ldots . (10)$$

which is a general statement of an equation now known as Stokes' Law. The original equation for the resistance or drag on small spheres as developed by Stokes (*1*) entirely on the basis of the forces of viscosity, was:

$$R = 3\pi\mu dv \ldots . (10)$$

By equating this equation and Newton's equation,

$$\left(R = C_D \frac{\pi d^2}{4}\frac{\rho v^2}{2}\right)$$

it becomes evident that:

$$C_D \frac{\pi d^2}{4}\frac{\rho v^2}{2} = 3\pi\mu dv$$

from which:

$$C_D = \frac{24\mu}{dv\rho}$$

or:

$$C_D = \frac{k}{\mathbf{R}}$$

Substituting a value of $k = 24$ in Eq 10:

$$v = \frac{1}{18}(g)\left(\frac{\rho s - \rho}{\mu}\right) d^2 \ldots (11)$$

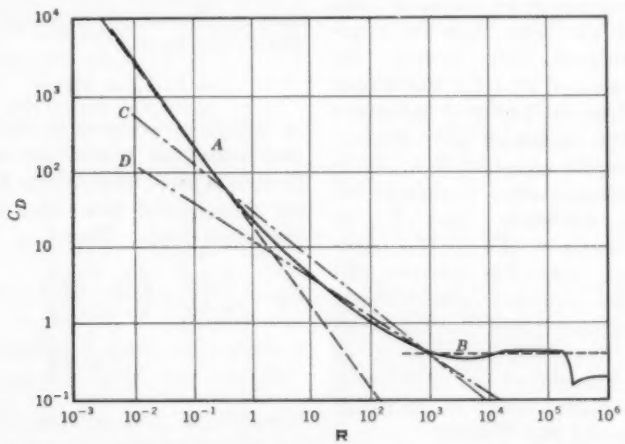

Fig. 1. Relationship of Reynolds Number to Drag Coefficient

The curves are for the following: A—*Newton's Law* ($C_D = 0.4$); B—*Stokes' Law* $\left(C_D = \frac{24}{\mathbf{R}}\right)$; C—*Allen's Law* $\left(C_D = \frac{30}{\mathbf{R}^{0.625}}\right)$; *and* D—*Allen's Law* $\left(C_D = \frac{12.65}{\mathbf{R}^{0.5}}\right)$.

Equation 11 is the usual form of Stokes' Law. Figure 1 shows that it fits experimental data exactly for values of **R** up to about 0.55, and offers a reasonably good fit up to **R** = 1.0.

Newton's Law

When inertia forces become so predominant that the accelerative and resistive forces are proportional to the second power of the velocity ($n = 2$), Eq 8 becomes:

$$C_D = k\left(\frac{vd\rho}{\mu}\right)^{2-2}$$

or $C_D = k$. Hence C_D is constant and $n = 2$, as originally assumed by Newton.

Substituting $n = 2$ in Eq 8:

$$v = \sqrt{\frac{4}{3}\frac{g}{k}d\left(\frac{\rho s - \rho}{\rho}\right)}$$

which checks Newton's Law as expressed by Eq 7.

From Fig. 1 it is evident that $C_D = k$ fits the experimental data in the **R** range of 1,000–250,000. Hence $C_D = 0.4$ and Newton's Law becomes:

$$v = 1.82\sqrt{\left(\frac{\rho s - \rho}{\rho}\right)dg} \quad . . (12)$$

Allen's Law

For transitional flow conditions, any value of n between 1 and 2 may be substituted in Eq 8 or 9. Taking an average of $n = 1.5$, it is evident from Eq 8 that:

$$C_D = k\left(\frac{vd\rho}{\mu}\right)^{1.5-2}$$

and:

$$C_D = \frac{k}{\sqrt{\mathbf{R}}}$$

From Eq 9:

$$v^{1.5} = \frac{4}{3}\frac{g}{k}\left(\frac{\rho s - \rho}{\rho}\right)\left(\frac{\rho}{\mu}\right)^{0.5}d^{1.5}$$

Taking the 1.5th root of both sides:

$$v = 1.21\left[\left(\frac{\rho s - \rho}{\rho}\right)\frac{g}{k}\right]^{\frac{1}{3}} \times \left[\frac{d}{\left(\frac{\mu}{\rho}\right)^{\frac{1}{3}}}\right] \quad . . (13)$$

which is a general statement of Allen's Law.

A study of the data in Fig. 1 reveals that when **R** = 10, $C_D = 4.0$, and when **R** = 1,000, $C_D = 0.4$. From the consideration that $\log C_D = \log k - 0.5 \log \mathbf{R}$, both of these conditions give a value of $k = 12.65$, and define the limits of application of Allen's Law to experimental data when $n = 1.5$. Therefore:

$$C_D = \frac{12.65}{\sqrt{\mathbf{R}}}$$

for all values of **R** from 10 to 1,000. Substituting $k = 12.65$ in Eq 13, Allen's Law becomes:

$$v = 0.22\left[\left(\frac{\rho s - \rho}{\rho}\right)g\right]^{\frac{1}{3}} \times \left[\frac{d}{\left(\frac{\mu}{\rho}\right)^{\frac{1}{3}}}\right] \quad . . (14)$$

For all values of **R** from 1.0 to 1,000, Allen's Law can be modified to give a reasonable degree of fit to the experimental data in Fig. 1. Solving simultaneously the equation $\log C_D = \log k - 0.5 \log \mathbf{R}$, for **R** = 1 and **R** = 1,000, it develops that for this range of **R** values:

$$C_D = \frac{30}{\mathbf{R}^{0.625}}$$

and $n = 1.375$. By substituting $n = 1.375$ and $k = 30$ in Eq 9:

$$v = 0.20\left[\left(\frac{\rho s - \rho}{\rho}\right) g\right]^{0.72} \times \left[\frac{d^{1.18}}{\left(\frac{\mu}{\rho}\right)^{0.46}}\right] \quad (15)$$

As generally used, however, Allen's Law is expressed in the experimentally modified form (which is somewhat more accurate than Eq 15:

$$v = 0.25\left[\left(\frac{\rho s - \rho}{\rho}\right) g\right]^{\frac{1}{3}} \times \left[\frac{d - 0.4d'}{\left(\frac{\mu}{\rho}\right)^{\frac{1}{3}}}\right] \quad (16)$$

in which d' is the diameter of particle at upper limit of Stokes' Law (0.0085 cm). This approximates Eq 14 as derived from the general expression in Eq 9.

Application of Laws

From the foregoing analysis it is evident that the coefficient of resistance, or drag, and consequently the settling velocity of particles, is a function of the Reynolds number. It is also evident that, from a knowledge of the variables involved in the resistance of a liquid to a moving particle, and of the exponent of velocity which obtains under various flow conditions, a general equation (Eq 9) can be developed. From such an equation, a series of specific formulations may be written such as Stokes' Law, Allen's Law, and Newton's Law which will conform reasonably well to experimental observations of sedimentation phenomena for all ranges of **R**, from 10^{-4} to 2.5×10^{8}.

The range of particle sizes to which each law is applicable can be determined by experiment. Table 1 lists the commonly accepted values for sand particles of specific gravity of approximately 2.65, along with a summary of other pertinent factors.

While Stokes' Law and Newton's Law are in excellent conformity with experimental data in certain ranges of **R**, there has long been dissatisfaction with expressions representing the very extensive zone of transition between the two. For example, the Allen's curve, derived directly from Eq 9 to represent **R** for values of 1–1,000, may give results which are in error by as much as 100 per cent. To overcome errors of this sort the range of values of **R** could be reduced and a whole series of equations set up, each of which would fit to a satisfactory degree a short segment of the experimental curve in Fig. 1. To use such curves, however, would become a very tedious process of elimination, because each involves a different odd power of velocity, and because both **R** and velocity are unknown for any particular diameter and density of particle with which the user might be concerned.

Because of this difficulty, a variety of equations has been formulated from experiments by a number of observers. Perhaps the best known was that developed by Allen Hazen (*2*) in his studies on sedimentation. It applied essentially to the same range of particle sizes as does Allen's Law. In general, the Hazen equation may be written:

$$v = 60.6\left(\frac{\rho s - \rho}{\rho}\right) d\left(\frac{T + 10}{60}\right) \quad (17)$$

in which v is velocity in millimeters

per second, ρs is the specific gravity of the solid, ρ is the specific gravity of the liquid (1.0 for water), d is the diameter of particle in millimeters, and T is the temperature of liquid in degrees F.

The simplicity of Hazen's equation makes it much easier to solve than Allen's and it is therefore very useful in many sedimentation problems. It does not, however, apply with sufficient accuracy throughout the transition zone.

Satisfactory methods of dealing with the transition zone between the limits of Stokes' and Newton's equations have recently been published by Fair and Geyer (*7*), who show the development of their equations and give examples of their use. Using methods of curve fitting, these authors have determined that a curve of equation

$$C_D = \frac{24}{R} + \frac{3}{\sqrt{R}} + 34 \ldots . (18)$$

fits the experimental data of Fig. 1 for **R** values of 0.5–10^4, effectively overlapping Stokes' Law at one extreme and Newton's Law at the other. To make use of this equation, a "settling-velocity term" is plotted against a "diameter term" for various values of **R**. The two terms are obtained by combining the equations for **R** and Newton's Law (Eq 7):

$$\mathbf{R} = \frac{\rho v d}{\mu}$$

and:

$$v = \sqrt{\frac{4}{3}\frac{g}{C_D}\left(\frac{\rho s - \rho}{\rho}\right)d}$$

From these:

$$v = \sqrt{\frac{4g}{3C_D}\left(\frac{\rho s - \rho}{\rho}\right)\frac{\mu \mathbf{R}}{\rho v}}$$

or:

$$\left(\frac{4R}{3C_D}\right)^{\frac{1}{3}} = \frac{v}{[g(\rho s - \rho)\mu]^{\frac{1}{3}}} \ldots (19)$$

and

$$\frac{\mathbf{R}\mu}{\rho d} = \sqrt{\frac{4}{3}\frac{g}{C_D}\left(\frac{\rho s - \rho}{\rho}\right)d}$$

or

$$(\tfrac{3}{4}\mathbf{R}^2 C_D)^{\frac{1}{3}} = \left[g\frac{(\rho s - \rho)}{\mu^2}\right]^{\frac{1}{3}} d \ldots (20)$$

The right-hand terms of Eq 19 and 20 are the "velocity term" and "diameter term," respectively.

An unusually direct and satisfactory method of finding settling velocities has been developed by Harold A. Thomas Jr. (*8*), who presents a single formula which is valid over the entire range of Stokes' and Allen's equations, and may be used for **R** values of 10^{-4}–10^3.

If both sides of Eq 7 are multiplied by $\frac{d\rho}{\mu}$, the resulting expression is:

$$\frac{dv\rho}{\mu} = \sqrt{\frac{4}{3}\frac{g}{C_D}\left(\frac{\rho s - \rho}{\mu^2}\right)\rho d^3}$$

By transferring C_D to the left hand side of the equation:

$$\sqrt{C_D} = \frac{dv\rho}{\mu} = \sqrt{\frac{4}{3}\, g\left(\frac{\rho s - \rho}{\mu^2}\right)\rho d^3}$$

or:

$$\mathbf{R}\sqrt{C_D} = \sqrt{\frac{4}{3} g\left(\frac{\rho s - \rho}{\mu^2}\right)\rho d^3}$$

Thomas has called the right hand portion of this equation "Allen's Number" (N_A). Hence:

$$N_A = \sqrt{\frac{4}{3} g\left(\frac{\rho s - \rho}{\mu^2}\right)\rho d^3} \ldots (21)$$

From the experimental data, the relation between **R** and N_A throughout the Stokes' and Allen's range is then:

$$\mathbf{R} = \frac{N_A^2}{24} \times \frac{1}{(1 + 0.035N_A)} \quad \ldots (22)$$

Substituting $\frac{dv\rho}{\mu}$ for **R**; Eq 21 for N_A; V_s for $\frac{1}{18} g \left(\frac{\rho s - \rho}{\mu} \right) d^2$, and solving for v, Eq 21 becomes:

$$v = \frac{V_s}{1 + 0.17 \sqrt{\frac{V_s d\rho}{\mu}}} \quad \ldots (23)$$

in which V_s is the velocity according to Stokes' Law.

Velocities can then be determined for particles of any diameter and density within the range of applicability of the equation by direct substitution, and without the necessity for determining a drag coefficient.

Equations in Table 1, as well as Eq 19 and 23 give settling velocities in centimeters per second when d is the diameter in centimeters, g is the acceleration of gravity (980 cm/sec^2), ρ is the density of liquid (1.0 g/cm^3 for water), ρs is the density of particle in grams per centimeter cubed, μ is the viscosity of liquid in poises (0.01309 at 10°C).

In applying the equations, use is generally made of Hazen's experimental evaluation of the relation between μ at 10°C and at any other temperature, T, expressed in °F. Hazen demonstrated that for water, at any degree F:

$$\mu = \mu \text{ at } 10°\text{C} \left(\frac{60}{T(°\text{F}) + 10} \right)$$

or:

$$\mu_T = 0.01309 \frac{60}{T + 10} \quad \ldots (24)$$

A similar equation developed by Poiseuille for μ at any degree C based on μ at 0°C states that:

$$\mu_c = \frac{0.0178}{1 + 0.03377Tc + 0.00022Tc^2}$$

Relation of Theory to Design

The best of the various laws of sedimentation conform quite well to experimental data for discrete spheres settling through still water in a basin of sufficient cross-sectional area that the influence of the walls is negligible, and when the concentration of suspended particles is sufficiently low that velocity field interference does not hinder settling. In any practical sedimentation basin, wall influence is of no importance. Velocity field interference depends upon concentration of particles. Fair and Geyer (*7*) have calculated the concentrations of various common materials necessary to reduce the settling velocity of particles by 1 per cent in the absence of flocculation or air entrainment to be approximately 6,000 mg/l for river silt, sewage grit, or precipitated $CaCO_3$. Settling velocity is approximately 2,500 mg/l for sewage solids. Comparing these values with the concentrations normally experienced in primary sewage settling or in water treatment indicates that velocity field interference should not be serious except perhaps near the bottom of the basin. On the other hand, variety in shape, size, and specific gravity of particles results in a wide range of settling velocities which, by causing some subsiding particles to overtake others, increases the natural tendency of suspended matter to flocculate. Thus, settling velocities change as subsiding material changes in size and density. Camp (*9*) estimates that

these velocities may be reduced by 10 per cent or more because of velocity field interference between particles, if the concentration of suspended particles is more than approximately 1,000 mg/l.

A number of factors greatly modify the settling velocity of particles in practical sedimentation basins. Specific gravity and shape of particles, coagulation or agglomeration of particles, concentration of particles, and or silt, or drastically reduced by organic matter or by entrained or adsorbed water and air bubbles. In addition, irregularly shaped particles settle more slowly than spherical particles of the same volume and material. Sand grains and heavy floc particles settle out quite rapidly in a practical sedimentation basin. They represent only a relatively small proportion of the particulate matter present, however. The result is that

TABLE 1

Summary of Laws and Ranges to Which Applicable

Law	Equation	Exponent of Velocity (n_1)	Relation of C_D to R	Range to Which Law Applies	
				Reynolds Number R	Particle Diameter ($\rho_s = 2.65$)
General Case	$v^{n_1} = \frac{4}{3}\frac{g}{k}\left(\frac{\rho_s - \rho}{\rho}\right)\mu^{n_1-2}d^{2-n}$	n	Variable	10^{-4}–2.5×10^5	Maximum limit not specified
Stokes'	$v = \frac{1}{18}g\left(\frac{\rho_s - \rho}{\mu}\right)d^2$	1	$C_D = \frac{24}{R}$	10^{-4}–0.55* 10^{-4}–1.0*	Up to 0.0085 cm* Up to 0.01 cm*
Allen's	$v = 0.22\left[\left(\frac{\rho_s - \rho}{\rho}\right)g\right]^{\frac{1}{2}}\left[\frac{d}{(\mu/\rho)^{\frac{1}{2}}}\right]$	1.5 1–2*	$C_D = \frac{12.65}{R^{0.5}}$	10–1,000	>0.01 cm <0.1 cm
Allen's Modified (Exper.)	$v = 0.25\left[\left(\frac{\rho_s - \rho}{\rho}\right)g\right]^{\frac{1}{2}}\left[\frac{d - 0.4d'}{(\mu/\rho)^{\frac{1}{2}}}\right]$	1.5 1–2*	$C_D = \frac{12.65}{R^{0.5}}$	10–1,000	>0.01 cm <0.1 cm
Allen's Modified	$v = 0.20\left[\left(\frac{\rho_s - \rho}{\rho}\right)g\right]^{0.72}\left[\frac{d^{1.18}}{\left(\frac{\mu}{v\rho}\right)^{0.45}}\right]$	1.375 1–2*	$C_D = \frac{30}{R^{0.625}}$	1–1,000	>0.01 cm <0.1 cm
Newton's	$v = 1.82\sqrt{dg\left(\frac{\rho_s - \rho}{\rho}\right)}$	2	$C_D = k = 0.4$	1,000–250,000	>0.1 cm

* Values shown are approximate.

movement of water through sedimentation tank are among the most important. Camp (*9*) notes that a turbid surface water may contain suspended matter ranging in specific gravity from about 2.65 for sand to about 1.03 for flocculated particles of organic matter and mud containing 95 per cent water, and that the floc particles resulting from coagulation may have a specific gravity of 1.18 for aluminum or 1.34 for iron oxides. These values may be increased by clay most of the particles settle at velocities within the range of Stokes' Law and the transition zone covered by Eq 19 and 23, or by the less accurate Allen's Law of Table 1.

Modifications of Stokes' Law to adapt it to flocculent or aggregated particles have been presented by Fair and Geyer (*7*). As they and other authors point out, however, the nature of flocculating suspended solids is so complex that the gap between theoretical and experimental findings and

their direct application to sedimentation tank design remains a broad one.

The effect of horizontal movement of water in continuous flow sedimentation basins, such as those commonly used in water and sewage treatment, is to cause settling velocities to differ significantly from the theoretical. A retarding effect on settling is produced by transportation and scour, which may be increased by turbulence. In many instances, however, turbulence and other velocity gradients may increase flocculation and thereby hasten sedimentation.

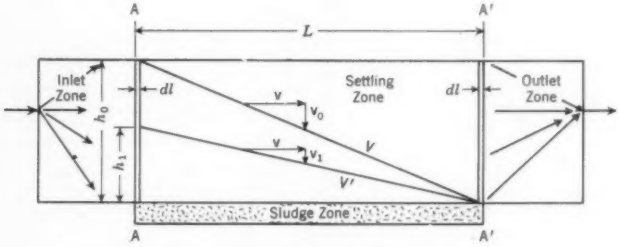

Fig. 2. Ideal Sedimentation Tank

A unit of water entering at left is assumed to spread out uniformly over plane A-A so that concentration of suspended particles is constant throughout incremental volume of tank.

Ideal Sedimentation Basin

Some design criteria have resulted from the analysis of an ideal basin by Hazen (*6*), Camp (*9, 11, 12*), Fair and Geyer (*7*), and others, although they are admittedly imperfect and leave much to be desired in translating theory into practical design criteria. Further experiment with inlet and outlet devices to approach more closely the ideal situation would contribute much to the efficiency of sedimentation basins.

Figure 2 may be used to illustrate what the ideal inlet and outlet device and the ideal tank should accomplish. A unit of water entering the ideal sedimentation basin at the left is assumed to spread out uniformly over a vertical plane A–A in such a manner that the concentration of suspended particles of all sizes and densities is constant throughout an incremental volume of the tank dL in length. The zone in which this takes place is called the inlet zone. This incremental volume of water moves from inlet to outlet at a uniform velocity (v) and arrives at the position A′–A′ without change in shape. The zone in which this occurs is called the settling zone. In the outlet zone, increments of water reassemble from all parts of the plane A′–A′ to reconstitute the original unit of water, carrying along all particles not removed in the settling zone.

Assuming that all particles settle discretely and that those particles which strike the bottom are removed, the absolute velocity path (V) represents the maximum elevation at which may be found particles of the smallest diameter d_0 which will experience 100 per cent removal. That is to say, a particle of diameter d_0 which enters

the basin at the water surface (at height h_0 above the bottom) will travel along the path V and be removed just as the water moving at velocity (v) enters the outlet zone. All other particles of the same diameter and density will follow parallel paths and be removed nearer the inlet end of the basin. The settling velocity (v_0) of the particle will, of course, be governed by its diameter, and density, and will follow the appropriate law of sedimentation.

Under practical conditions, particles which strike the bottom of a tank may roll along the bottom instead of being removed. The horizontal velocity at which this occurs is known as the critical velocity. Available information indicates that in a properly designed basin the ratio of the mean horizontal velocity in the tank to the settling velocity of the smallest particle which will be removed should not exceed 20:1 to 40:1.

Particles of diameter (d_1)—less than d_0 but of the same density—will follow an absolute path parallel to V'. Consequently they will be carried over the outlet if they originate above h_1 in a plane A–A, or be removed if they enter the tank at a lower elevation.

Because the ratio of v_0 to v_1 equals the ratio of h_0 to h_1, it may be concluded that the percentage removal of particles smaller than d_0, but of the same specific gravity, will be:

$$\% \text{ removal of diameter } (d_1) = \frac{v_1}{v_0} \times 100 \quad (25)$$

in which v_0 and v_1 are determined by the appropriate law of sedimentation.

The time t required for a particle of diameter d_0 to settle through a depth h_0 at velocity v_0 is:

$$t = \frac{h_0}{v_0}$$

or:

$$v_0 = \frac{h_0}{t}$$

Because t is also equal to the basin volume divided by the inflow rate Q, and volume equals the surface area A times h_0,

$$t = \frac{Ah_0}{Q}$$

Then:

$$v_0 = \frac{h_0}{Ah_0} \div Q = \frac{Q}{A}$$

or, by Stokes' Law:

$$d_0 = k(v_0^{\frac{1}{2}}) = k\left(\frac{Q}{A}\right)^{\frac{1}{2}}$$

Hence it is apparent that, theoretically, the smallest diameter of particle which will be completely removed is a function of the surface-loading or overflow rate rather than of the depth of the tank when the horizontal velocities in the basin are approximately uniform. This is true for non-agglomerating particles, such as silt, when no velocity field interference occurs, but may not be absolutely correct for floc particles which may agglomerate while settling.

It is obvious, of course, that inlet devices are incapable of distributing a unit of water uniformly over a transverse cross section of the sedimentation basin. Similarly, a unit of water leaving the basin does not represent a reassemblage of water so distributed. Such a failure of inlet and outlet devices to approach the ideal situation, together with other phenomena, lead to a lack of uniformity of horizontal velocities of such nature that many engineers question the validity

of the surface-loading criterion in practical tank design.

Conclusion

The laws of sedimentation of discrete particles of spherical or approximately spherical shapes are well established as a result of theoretical and experimental studies. To an appreciable degree they have been modified to account for flocculation of particles. With some degree of accuracy their application to a simplified ideal sedimentation basin is possible. The failure of practical basins to approach the ideal, however, creates complex situations which have delayed the time when sedimentation basins will be designed entirely by rational procedures.

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Fluoride Distillation Method Using Controlled Air Jet

R. L. Morris and Josephine Cerny

A contribution to the Journal by R. L. Morris, Chief Chemist, and Josephine Cerny, Assoc. Chemist, both of the State Hygienic Lab., Iowa City, Iowa.

A REAL need exists for a simple, rapid, and precise method for separation of fluoride ions from interfering substances in natural water. The majority of these interferences are nonvolatile and distillation is possible because fluorides can be volatilized from highly acid solutions and collected by condensation. Numerous methods have been proposed for controlling thermal levels by steam distillation or by carefully administered direct-flame distillation methods.

Previous Methods

The steam distillation procedure, as described in *Standard Methods* (*1*), frequently produces highly yellow-colored distillates when applied to polluted natural river waters. This color seriously interferes with both visual and photometric evaluation, especially of waters with a low level of fluoride. The need for rather constant supervision of the two-burner thermal application to the steam and distillation flasks, coupled with the cumbersome equipment required by the steam distillation technique, has rendered this procedure quite unsuitable for multiple analysis of large numbers of samples.

Thrun (*2*) proposed a high turbulence created by air flow through the sample, along with simultaneous rapid distillation by direct flame. This method was fast and provided acceptable results when carefully done, but it did not specify air flow or distillation rates. Considerable deviation was encountered because of physical carry-over of acidity and sulfates into the distillate when the Thrun method was used routinely. If an attempt to find a method for precisely controlling air flow and ascertaining the optimum distillation rate, it was decided to investigate the distillation characteristics.

Distillation Apparatus

Several homemade air-flow meters were tested and were found to work successfully. Because of the individuality of these manometers, however, each of them had to be calibrated separately. A commercial flow-meter * with a stainless steel ball was finally used because of its simplicity and convenient size. These flowmeters are precisely made and can be used interchangeably at the same setting, which makes them especially applicable for multiple analysis. The flowmeter, as used in the experiments reported here, was preceded by an air scrubber consisting of a liter flask with distilled water. Figure 1 is a schematic diagram of the distillation train, and the actual equipment is shown in Fig. 2.

* Equipment used was an "Air CFH" flowmeter, product of The Dwyer Co., 317 S. Western Ave., Chicago, Ill.

The distillation flask consisted of a 300-ml long-neck Kjeldahl flask with a specially constructed side jet climaxing at a 45-degree angular position ¼ in. above the bottom of the exact center of the flask. A 1-mm opening existed in the jet which was fabricated from 4-mm glass tubing. The same effect could be achieved by inserting a 4-mm glass tube through the rubber stopper with the described angular jet opening.

Distillation Procedure

Transfer a 100-ml portion of sample, or an appropriate aliquot diluted to 100 ml with distilled water, plus 10 ml of distilled water into the air-jet distillation flask and add 0.2–0.3 g of silver sulfate and a few glass beads. Then add 40 ml of concentrated, fluoride-free sulfuric acid, pouring it down the neck of the flask with a minimum of mixing. Connect the distillation train and start the air flowing (air-flow rate was 4.5 units on the air-flow-meter used in these experiments). Allow air to flow for about ½ min and then apply heat to distill at a rate of 10 ml per minute. Collect 90 ml in the graduate, dilute to 100 ml with distilled water, and mix. Titrate 10 ml of this distillate with 0.02*N* sodium hydroxide and calculate the acidity in parts per million. If less than 1 ml of 0.02*N* alkali is used, the acidity is less than 100 ppm and the distillation is acceptable. The titration is the simplest

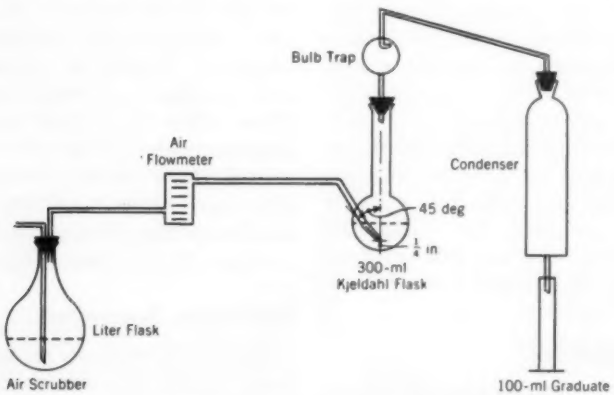

Fig. 1. Schematic Diagram of Distillation Train

The fluoride distillation apparatus is shown here as it was used in the tests. Although several experiments were successfully run with homemade air flowmeters, a commercial flowmeter was finally used for the tests.

A glass bulb trap was used between the distillation flask and the water-jacketed condenser to reduce entrainment of acid in the distilling vapor. A common vertical condenser led into a 100-ml graduate for collection of the distillate.

An asbestos-cement plate with a 7-cm hole was fitted around the bottom of the flask to restrict heat from the sides of the vessel. Heat was furnished by a Fisher burner. The air source was the house compressed-air supply.

parameter by which the efficiency of the distillation can be measured. This titration parameter assures the analyst, without question, that an efficient distillation has taken place which is a valuable essential for referee work. Statistically, results show that measurable sulfate carryover occurs solely in waters highly contaminated with organic matter, and even then only in less than 1 per cent of these exaggerated cases. The titration, which consumes approximately 1 min of time, should therefore be considered only as a safety factor and not an integral part of the actual distillation process. The entire distillation procedure, including the acidity titration, requires about 12 min.

The 90-ml remainder of the distillate can be analyzed for fluoride ion content by any suitable colorimetric method. The analytical results in this study were determined using the visual Scott-Sanchis modification described in 9th edition of *Standard Methods* (*3*).

Experimental Data

Previous attempts to use air turbulence and rapid distillation for fluoride volatilization gave unacceptable accuracy and precision, and experiments were undertaken to find the optimum distillation rate in regard to thermal application and air flow.

It was determined that excessive air turbulence resulted in physical entrainment of the sulfuric acid in the vapor bubbles released at the liquid surface and gave acidities and sulfate contents of unacceptable magnitude in the distillate. Sulfate contents of greater than 500 ppm were frequently found in the distillate when uncontrolled air-flow rates were used. This highly contaminated distillate was obviously unsatisfactory for subsequent colorimetric fluoride evaluation. An air-flow rate of 4.5 units on the flowmeters used gave optimum fluoride recovery with minimum acidity and sulfate carryover.

Studies on distillation rates demonstrated that volumes of 10 ml per minute gave the most precise and accurate recovery of fluoride ions in the distillate when a 4.5-unit air-flow rate was used.

TABLE 1

Recovery of Fluoride Increment From Distilled Water

Fluoride Added *ppm*	Fluoride Recovered *ppm*	Deviation *ppm*
0.0	0.0	0.0
0.0	0.0	0.0
0.0	0.0	0.0
0.5	0.5	0.0
0.5	0.55	0.05+
0.5	0.5	0.0
1.0	0.95	0.05−
1.0	1.0	0.0
1.0	1.0	0.0
1.5	1.5	0.0
1.5	1.6	0.10+
1.5	1.6	0.10+
2.0	2.10	0.10+
2.0	2.0	0.0
2.0	2.0	0.0
2.8	2.83	0.03+
5.0	5.0	0.0
5.0	5.3	0.3+
5.0	5.2	0.2+

Also, use of the bulb trap between the distillation flask and the condenser markedly reduced carryover of nonvolatile impurities at high distillation rates. Large numbers of replicate distillations have shown that careful work can produce distillates having less than 50-ppm acidity and sulfates. It was the practice in these tests to discard as unsatisfactory any distillate evidencing greater than 100-ppm acidity. Such acidities occur only rarely with experienced operators.

A series of standard sodium fluoride samples prepared with distilled water were evaluated using the above described air-jet distillation apparatus, and the results are given in Table 1. A distillation blank of 0.1 ppm for 100 ml of distillate was used in this entire investigation. Excellent recovery of the fluoride increment from distilled-water standard solution was evidenced with analytical precision among the replicates of each concentration level.

All samples of more than 0.8-ppm fluoride content were analyzed using a 50-ml or smaller aliquot of the distillate and the appropriate factor. This approach influences the precision as well as the effect of the distillation blank which becomes insignificant at magnitudes above 0.8 ppm. It is suggested, where possible, that the sample aliquot for distillation be chosen so that distillate portions analyzed be at least 25 ml.

A sequence of samples was prepared using Iowa River water and increments of sodium fluoride to evaluate recovery of fluoride ion from a natural water of appreciable mineralization. The results of the air-jet distillation procedure on these samples are shown in Table 2. Maximum deviations of 0.1 ± are evidenced by these data.

Water samples from several other rivers and wells were used to prepare fluoride increment sequences. Analytical results obtained by the air-jet distillation method were comparable to Ohio River water test results with respect to both accuracy and precision.

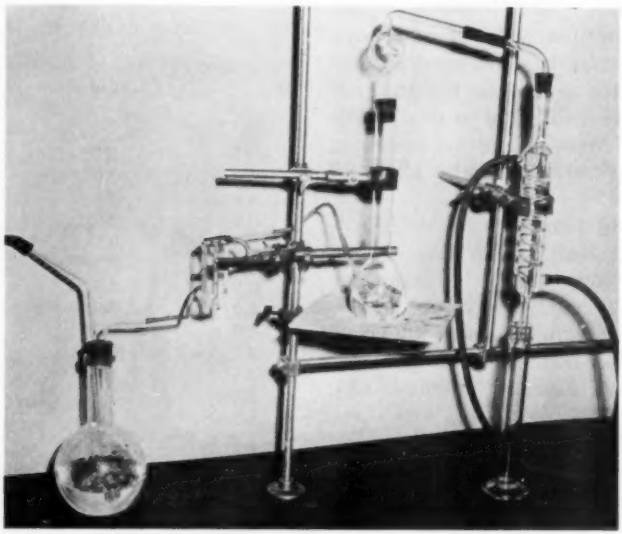

Fig. 2. Apparatus for Fluoride Distillation

The need for a rapid and precise method of fluoride distillation becomes greater as polyphosphate materials are more commonly used.

An industrial-waste sample of high fluoride content was analyzed by non-distillation and air-jet distillation procedures. A dilution ratio of 1 to 100 was used for the non-distillation procedure which eliminated, by selective dilution, adverse effects of naturally occurring interferences in the original sample. Replicate results gave 74 ppm (F^-).

A 100-ml portion of the original sample was subjected to air-jet distillation and replicate results, using 1-ml portions of the collected distillate, gave 75 ppm (F^-).

This tends to indicate that fluoride ion magnitudes of at least 75 ppm can be quantitatively distilled from natural waters by the air-jet technique.

Discussion

Data listed in Tables 1 and 2 show that effective recovery of fluoride increments from both distilled-water standards and natural waters can be accomplished by the air-jet distillation procedure. Preliminary experiments demonstrate that fluoride levels as high as 75 ppm can be volatilized quantitatively.

It is felt that laboratories desiring to use this technique should run an acidity carryover curve to determine the optimum air flow-rate setting on their own flowmeter which could then be used to calibrate other flowmeters for replicate apparatus. Data from the present tests show that distillates can average as low as 20-ppm acidity and sulfate with carefully controlled air flows. Distillation flasks can be constructed by any glass blower and they can be used interchangeably if the design described above is duplicated.

Correlative work on natural river waters evidencing yellow-color carryover in the distillate by steam distillation procedures invariably produced colorless and acceptable distillates when the air-jet distillation technique was used.

Recovery of the entire sample without dilution eliminates multiplication errors and reduces the necessity for concentration of samples low in fluoride.

Waters containing large amounts of organic matter must sometimes be subjected to preliminary ashing because they exhibit excessive turbulence. Subsequent carryover, indicated by the acidity titration, renders the distillate unfit for analysis.

The air-jet distillation method lends itself to routine work, because an efficient analyst can successfully handle several distillations simultaneously. The commercial flowmeters can be set once during the early part of the distillation and after that they require no further attention.

TABLE 2

Recovery of Fluoride Increment From Iowa River Water

Fluoride Added *ppm*	Total Fluoride* *ppm*	Fluoride Recovered *ppm*	Deviation *ppm*
0.4	0.6	0.6	0.0
0.4	0.6	0.6	0.0
0.4	0.6	0.6	0.0
0.6	0.8	0.8	0.0
0.6	0.8	0.85	0.05+
0.6	0.8	0.8	0.0
1.0	1.2	1.2	0.0
1.0	1.2	1.3	0.1+
1.0	1.2	1.2	0.0
1.4	1.6	1.6	0.0
1.4	1.6	1.6	0.0
1.8	2.0	2.1	0.1+
1.8	2.0	1.90	0.1−
1.8	2.0	2.0	0.0
5.0	5.2	5.2	0.0

* Fluoride content of Iowa River water as determined by triplicate distillation procedures was 0.2 ppm.

The increasing use of polyphosphate materials in municipal and industrial installations magnifies the need for such a rapid and accurate fluoride technique because of the marked interference of this class of compounds.

Many analysts have turned away from previous fluoride distillation procedures because of the tedious nature of the technique. The rapid and simple controlled air–jet modification described here should encourage laboratories to undertake more frequent primary-interference separation in fluoride analysis.

Acknowledgment

The authors wish to acknowledge the valuable assistance and helpful suggestions of Bruce D. Andrews and Laurence A. Devine in the early stages of development of this technique.

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Comparative Fluoride Analyses by Several Methods

Joseph V. Thom and George P. Gribkoff

A contribution to the Journal by Joseph V. Thom, Assoc. Public Health Chemist, Sanitation Lab., State Dept. of Public Health, Berkeley, Calif., and George P. Gribkoff, Assoc. Public Health Chemist, Branch Public Health Lab., State Dept. of Public Health, Los Angeles, Calif.

AN impression, gained from discussion with treatment plant operators, chemists, and engineers interested in the fluoride content of water, indicates that two major improvements in the laboratory determination of fluoride in water supplies are needed. They are: [1] a quick and simple method for determining fluoride, and [2] a method that will permit extreme accuracy in determining fluoride.

Many methods, or modifications of methods, designed to achieve either of these needs have appeared in the literature, but there is a notable lack of published data comparing values obtained by actual analysis of natural waters by the various methods proposed. This article is presented with the hope that it may stimulate further laboratory comparison of available methods in order to estimate objectively their reliability.

Analyses by each of four published methods were used in the course of this study. The methods were: the *Standard Methods* procedure (*1*), hereafter referred to as the "standard method," which is a modification of the well known Sanchis method; the method developed by Maier and Megregian (*2*), which is hereafter referred to as the "M and M method"; the method of W. E. Thrun (*3*), which will be referred to as the eriochrome method; and the method reported by Rubin (*4*), which will be referred to as the "rapid method."

Principles of Methods

Principles of the above-mentioned methods will be described generally.

The standard method makes use of the alizarin-zirconium lake, which gives a water-soluble red-colored solution. In the presence of fluoride ion, zirconium is removed from the lake to form zirconium fluoride, thus lessening the intensity of red color of the solution. As the concentration of fluoride increases, the color of the solution becomes less red, and begins to turn yellow. A visual comparison is made between unknown samples and simultaneously prepared standards.

The M and M method employs the same principle as the standard method, but the concentration of alizarin and zirconium in the reagent preparation and the acidity of the solution adapt the procedure to make possible a photometric measurement of the color intensity. The rapid method also uses the same chemical principle as the standard method, but the concentration of alizarin and zirconium and the acidity of the reagent solution are designed to give suitable readings in the range of 0.6–1.4 mg/l of fluoride ion after a 5-min reaction time. Readings are

visual and are compared to artificial standards. The standard method and the M and M method require a 1-hr reaction time. The principle of the eriochrome method depends upon the destruction of the reddish-purple color of the aluminum-eriochrome cyanine lake by fluoride ion, which reacts with the aluminum to form aluminum fluoride. As the fluoride ion in the mixture increases, the color of the solution changes from the reddish-purple to red, to cherry red, to peach pink, and finally to orange. Readings are by visual comparison against simultaneously prepared standards after a minimum reaction time of 10 min.

Presentation of Data

The results of the analyses of natural water samples from different sources are given in Table 1. For statistical treatment of these results, it was convenient to group them according to the level of the fluoride concentration found. The results by the standard method were used for the purpose of grouping.

The mean values expressed in Table 1 are followed by the expected deviation as determined by replicate analyses on prepared standard samples. The analytical data expressed in Table 1 was not originally collected for the purpose of statistical comparison and, to make such a comparison, an independent means of estimating the deviation of the results obtained by each method had to be established. This was done by replicate determinations on prepared samples. To avoid difficulties in treating the data expressed in Table 1, the mean of the values found by each method is compared and the deviations found on prepared samples are used to express their expected variability.

The prepared standard samples were large-volume solutions of sodium fluoride with replicate portions dispensed in polyethylene bottles. These bottles were given code numbers and distributed among ten chemists for analysis. No interfering substances were added to these samples. The results of the analyses are shown in Fig. 1 and a statistical summary is given in Table 2.

These samples were prepared at three different fluoride concentrations because of the possibility of a variation of deviation errors with fluoride concentration. Levels of 0.3 mg/l, 0.8 mg/l and 1.2 mg/l were chosen because they best covered the range which is of interest in water treatment practice.

Validity of Comparison

The samples of known fluoride content were used to establish the standard deviation of each method in the absence of interferences. The application of deviations calculated from these data to the mean values obtained for the natural water samples is valid only if it is assumed or shown that no interfering substances were present in the natural water samples. Comparison of the mean values in Table 1 shows that results by each of the methods are comparable within the deviations found on the prepared standard samples which contained no interferences. This shows that, as concerns these natural waters, interfering substances did not significantly affect the results. This reasoning does not take into account the possibility that an interfering substance would affect the results equally in each method. The facts established by other investigators gener-

TABLE 1

Analyses of Natural Waters

Sample Number	Fluoride Concentration—*mg/l*		
Group A			
	Standard Method	M and M Method	Eriochrome Method
1,636	0.30	0.40	0.35
1,637	0.40	0.44	0.65
1,638	0.30	0.34	0.50
1,639	0.30	0.34	0.40
1,640	0.20	0.28	0.40
1,723	0.25	0.20	0.50
1,724	0.35	0.40	0.60
1,736	0.30	0.40	0.60
1,737	0.30	0.40	0.60
1,738	0.43	0.50	0.40
1,739	0.44	0.44	0.45
1,741	0.40	0.44	0.40
1,746	0.27	0.20	0.35
1,752	0.30	0.32	0.30
1,753	0.25	0.24	0.20
1,762	0.40	0.50	0.40
1,946	0.16	0.08	0.17
1,947	0.27	0.22	0.45
2,033	0.20	0.10	0.37
Mean value	0.31	0.33	0.43
Expected deviation	±0.05	±0.06	±0.06
Group B			
	Standard Method	M and M Method	Eriochrome Method
1,754	0.50	0.38	0.60
1,774	0.90	0.90	1.00
1,945	0.53	0.48	0.62
1,949	0.77	0.78	0.85
1,951	0.52	0.54	0.70
Mean value	0.63	0.62	0.75
Expected deviation	±0.03	±0.04	±0.06

Sample Number	Fluoride Concentration—*mg/l*		
Group C			
	Standard Method	M and M Method	Eriochrome Method
1,742	1.00	1.00	0.90
1,743	1.20	1.24	1.30
1,744	1.60	1.68	1.80
1,745	2.00	2.08	2.50
1,750	1.30	1.14	1.40
1,761	1.10	1.14	1.10
1,775	1.30	1.48	1.20
1,776	1.10	1.18	1.20
Mean value	1.32	1.37	1.42
Expected deviation	±0.06	±0.04	±0.06
Group D			
	Standard Method	M and M Method	Rapid Method
431	1.0	0.97	0.7
432	1.2	1.20	0.9
529	1.0	1.14	0.8
530	1.20	1.20	0.8
479	0.9	1.08	0.6
480	0.9	1.00	0.6
647	1.2	1.13	0.8
648	1.0	1.14	0.7
650	1.0	1.03	0.7
774	1.0	0.80	0.7
773	1.0	0.80	0.6
957	1.0	0.90	0.7
956	1.0	0.90	0.8
1,263	1.0	0.96	0.8
1,282	0.8	0.69	0.4
1,401	0.8	0.78	0.6
1,402	1.0	1.00	0.8
Mean value	1.00	0.98	0.7
Expected deviation	±0.06	+0.04	±0.4

ally eliminate the probability of this occurrence. This point is expanded below.

Interferences

When interferences are present in a sample, their effect upon each of the methods is different. The reagent responsible for the color development in each method is either chemically different from that of another method or is at a different concentration (*2, 5*). The data given in Tables 3 and 4 illustrate the effect of interferences on two methods which differ in the concentration of color-forming reagent.

Detection of Interferences

The data of Tables 3 and 4 indicate a practical approach to the problem of interferences in laboratories which receive many routine samples for fluoride determinations from widely different sources. In essence, the problem is one of detecting interferences without resorting to distillation or complete analysis of all samples. The use of two methods, such as the standard method and the M and M method, would indicate the presence of interferences and necessity for distillation if the two results were not in agreement within the limits of the expected deviations.

An additional system that may be used is that of diluting a portion of a sample to half strength with distilled water, and then determining fluoride on the diluted and undiluted sample. The assumption is made that, in addition to diluting the fluoride concentration to half strength, the interferences, if present, are decreased in strength, possibly to below the point of interference. The fluoride content of the diluted and undiluted samples should correspond within the expected deviation of the method after the dilution factor has been applied to the calculation. This is possible because the effect of interferences is not a linear function of concentration and usually has a threshold value for most interfering ionic species (*2, 5*). Noncorrespondence of the diluted and undiluted sample portions with regard to fluoride content indicates the presence of interferences and the necessity for distillation.

TABLE 2

Summary of Data on Solutions With Known Fluoride Concentrations

Method	Mean Value by Analysis—mg/l	Standard Deviation	Coefficient of Variation—%
Known Value: 0.30 mg/l Fluoride			
Standard	0.31	±0.05	16.1
M and M	0.28	±0.06	21.4
Eriochrome	0.27	±0.06	22.2
Rapid	Not applicable		
Known Value: 0.80 mg/l Fluoride			
Standard	0.81	±0.03	3.7
M and M	0.78	±0.04	5.1
Eriochrome	0.84	±0.06	7.2
Rapid	Not applicable		
Known Value: 1.20 mg/l Fluoride			
Standard	1.22	±0.06	4.9
M and M	1.20	±0.04	3.3
Eriochrome	1.23	±0.06	4.9
Rapid	0.93	±0.43	46.2

Discussion

From a laboratory point of view, the M and M method has a definite advantage because a complete set of

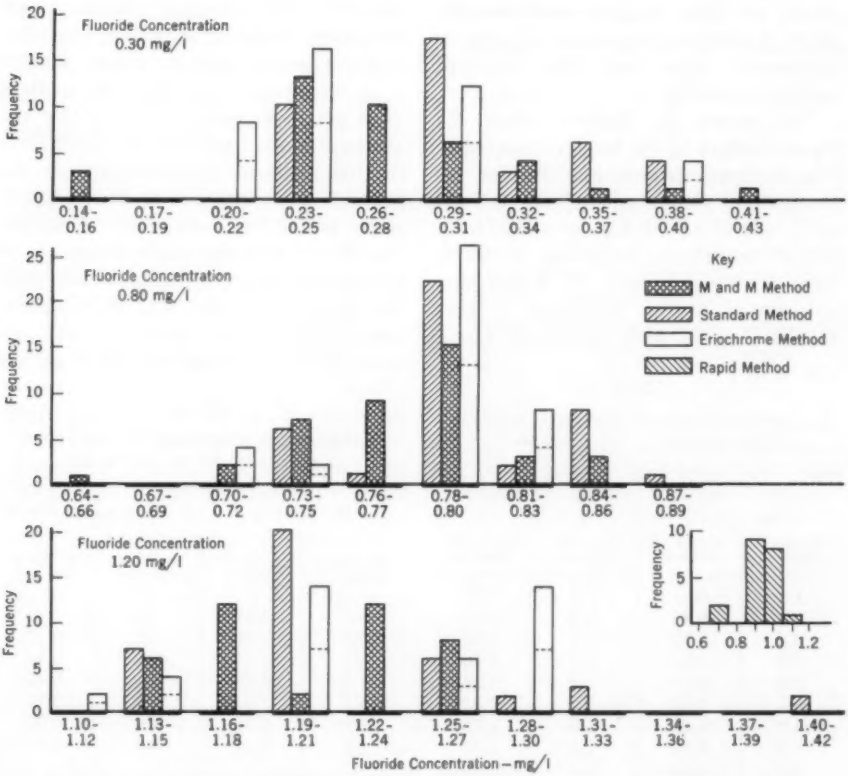

Fig. 1. Replicate Analyses of Three Prepared Solutions With Known Fluoride Concentrations

For standard method and M and M method, 40 analyses were made at each fluoride level. Only 20 analyses at each level were made with rapid method and eriochrome method. The frequency of values found for the eriochrome method were doubled for better comparison, and the true values are shown by the dividing line at the center of each bar. The rapid method was inapplicable at the 0.30- and 0.80-mg/l levels.

standards need not be set up for each fluoride determination once the standard curve is constructed. A periodic check should be made by running several standards simultaneously. The data in Tables 3 and 4, however, indicate a disadvantage of the M and M method in that the presence of interfering substances produces a very much larger degree of error.

Consideration of the necessary equipment for performing each of these methods of analysis would be a definite factor in deciding which method to use. The standard method, by comparing color and color intensity in Nessler tubes, is, in a sense, a more direct method of comparison. For this reason, some water works personnel who are not familiar with photometric tech-

niques or whose budgets preclude purchase of relatively expensive laboratory equipment, may find the standard method desirable.

The results in Table 1 show the rapid method to be largely unreliable. The authors suspect that further investigation would prove this method to be useful in the hands of one individual repeatedly analyzing a water from a single source. It is felt that adjustment of the standards to give reliable results with waters of essentially constant chemical characteristics is possible, but it may require a greater knowledge of chemical procedures than nontechnical users of the method have employed.

TABLE 3

Replicate Determinations by Different Chemists on 1.25mg/l Sample *

Not Distilled		Runs Individually Distilled	
Standard Method	M and M Method	Standard Method	M and M Method
1.0	0.51	1.2	1.16
0.8	0.45	1.2	1.34
0.8	0.50	1.4	1.16
1.0	0.55	1.3	1.36
0.8	0.48	1.0	1.24
0.8	0.53	1.3	1.33
0.9	0.42	1.3	1.29
1.0	0.56	1.5	1.17
0.9	0.49	1.3	1.20
0.9	0.54	1.2	1.31

* Sample contained 1.25 mg/l fluoride ion and 5 mg/l aluminum ion.

Conclusions

For all the methods studied, the results of the analyses of the known solutions indicate agreement within the standard deviation among the mean values obtained by each method at each of the fluoride ion concentrations considered. The standard deviation for the rapid method is so large, however, and the mean value so widely variant from the known value, that this method does not give useful results under the circumstances used for its evaluation in this instance. The results on the natural water samples indicate agreement within the expected deviations, but the eriochrome method appears to give a consistently higher result than the other two methods at all the levels considered. The coefficient of variation is the standard deviation of the distribution expressed as a percentage of the mean of the distribution. All of these methods, by consideration of the coefficient of variation, have a definitely comparable degree of precision and accuracy at each level. In the low-level fluoride concentration range the coefficient of variation is relatively large for all of the methods.

TABLE 4

Replicate Determinations by Different Chemists on 0.75mg/l Sample *

Not Distilled		Runs Individually Distilled	
Standard Method	M and M Method	Standard Method	M and M Method
1.0	2.61	0.8	0.73
0.8	2.68	0.6	0.78
1.2	2.71	0.8	0.75
1.0	2.40	0.7	0.88
1.0	2.64	0.8	0.72
1.0	2.59	0.8	0.79
1.0	2.62	0.8	0.77

* Sample contained 0.75 mg/l fluoride ion and 20 mg/l phosphate ion.

The authors believe that the data presented conclusively show that two of these methods may be used in lieu of the *Standard Methods* procedure in water works practice. From the obviously poor results obtained with the

rapid method, it appears that it is not reliable for a laboratory receiving waters from many different sources for routine fluoride determination.

Acknowledgments

The authors wish to express their appreciation to the chemists of the staff of their organization for performance of the analytical work and to William Holy of the US Public Health Service, Dallas, Tex., for some of the analytical data presented. The authors are indebted for the stimulus provided by the interest of Franz J. Maier, Stephen Megregian, and Harry P. Kramer, which stems from their diligent investigations of fluoride problems for the US Public Health Service.

References

1. *Standard Methods for the Examination of Water and Sewage.* APHA & AWWA, New York (9th ed., 1946), p. 76.
2. Megregian, Stephen & Maier, Franz J. Modified Zirconium-Alizarin Reagent for Determination of Fluoride in Water. *Jour. AWWA,* **44**: 239 (1952).
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4. Rubin, Lazarus. A Rapid Method for the Determination of Fluorides in Water Supplies. *J. NEWWA,* **66**:1 (1952).
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Standards for Deep Well Vertical Turbine Pumps

Committee Report

A committee report presented on Jun. 15, 1955, at the Annual Conference, Chicago, Ill., by M. H. Owen, Co-Chairman, ASA Committee B58—Deep Well Vertical Turbine Pumps. Other AWWA members of the committee were: C. D. Bower, W. L. Dornaus, Fenmore Dunn, N. C. Ebaugh, C. R. Erickson, Edward Farmer (deceased), A. D. Henderson, H. A. Knudsen, T. R. Mackay, M. S. McIlroy, L. E. Ordelheide, J. L. Perhab, and Dan Rankin. (P. H. Brown, Co-Chairman, represents ASME.)

IN 1949, a committee consisting of Perry H. Brown and Marvin H. Owen as co-chairmen, together with 22 other members, representing users, manufacturers, consultants, and engineers, was assigned the task of developing standard specifications for deep well vertical turbine pumps for the ASA under the sponsorship of the AWWA.

On receiving the assignment, the committee investigated previous work in this field. Research disclosed that a committee had been previously established for the same purpose, but had apparently abandoned the task as hopeless. The National Association of Vertical Turbine Pump Manufacturers had developed a standard but it was incomplete and had the acceptance of neither pump users as a whole nor the ASA. It did, however, assist greatly in the task before the committee.

The first activity of the new group was the formation of a working committee, all the members of which were in the vicinity of Los Angeles. This group, which consisted of Perry H. Brown, Dan Rankin, Wilson L. Dornaus, Aladar Hollander, Clark D. Bower, Thomas R. Mackay, John L. Perhab, and Marvin H. Owen, held a series of meetings and listed the desirable points to stress in such a standard.

Preliminary Efforts

In investigating the more popular existing deep well vertical pumps it was found that eight different column lengths were used, namely: 20 ft, 16 ft, 15 ft, 12 ft, 10 ft, 9 ft-11⅝ in., 6 ft, and 5 ft; and almost as many threads, including both straight and tapered. For oil tubing it was found that standard black iron pipe, extra-heavy pipe, and tubing were all commonly used, with each manufacturer setting up individual systems for shaft guides and spiders. No uniform method was used to determine straight wells or pumping-head reference points. Even methods of determining column loss or entrance loss varied greatly. One manufacturer, in his published report, used velocity head three different times in presenting a curve on a given pumping unit.

In view of these problems, the assignment at first appeared quite discouraging. After the first year, however, it seemed somewhat more hopeful; the working committee sent questionnaires to the other members with the request that they, in turn, acquaint themselves with the opinions of both users and manufacturers in their localities. The results of this effort were quite helpful, for it was soon apparent that the users were greatly in favor of standardization and, in particular, of a uniform 10-ft column length, so that a single rig could be used both for installation and maintenance. In this regard, however, a difference of opinion existed between users and manufacturers. Finally, in 1954, the vote was so overwhelmingly in favor of 10-ft column lengths that this specification was accepted. One of the larger pump builders, on hearing of this vote, voluntarily changed its long established use of 12-ft sections to the 10-ft standard.

Many other features which the committee desired to adopt had either to be settled by compromise or left out altogether in order that the group's work could arrive at the present stage of development—the section on field testing being a case in point.

In determining a standard for shafting, the only previous effort available was the ASA Code B17c for the design of transmission shafting, which had been sponsored by the ASME. By the time AWWA's tentative standard was ready to be submitted, the ASME had requested that the ASA withdraw its standard on the grounds that it was no longer necessary, referring apparently to the use of line shafting for belt-driven equipment and overhead installation. This B17c standard is based on 1020 steel, which is not used to a great extent by the industry at present but does cover the vertical turbine pump field with a good safety factor; and if other material (such as 1045 steel) is used, the factor of safety is still generous.*

It would be amiss not to mention at this time the submersible-type pumping unit. Because of the rapid development of this type of pump and its application to highly developed areas, the committee believes a section on the subject should be added to the standard. A tentative specification covering such units has been drafted, and it is hoped that in the not too distant future a field test code will also be submitted.

Specifications

The committee's specifications were approved by the ASA on May 11, 1955 and published by the AWWA in July of that year (*1*). Specification B58.1 is recommended as a guide for the deep well turbine pump user in selecting new equipment. The suggested standards are to be considered as a minimum requirement for a first-quality deep well turbine pump, but do not preclude the use of more elaborate specifications on the part of either user or manufacturer; nor is it intended to restrict the use of any equipment not meeting the requirements of this specification, if the user does not consider such compliance necessary.

This specification is applicable primarily to pumps that are constructed of accepted standard materials of the best quality and workmanship, designed to handle cold, clear water, usually from an underground well.

* As portions of B17c are referred to in the B58.1 specifications, the eventual status of B17c came into question. AWWA sponsorship of that part of B17c which applies to deep well vertical pumps is now under consideration.

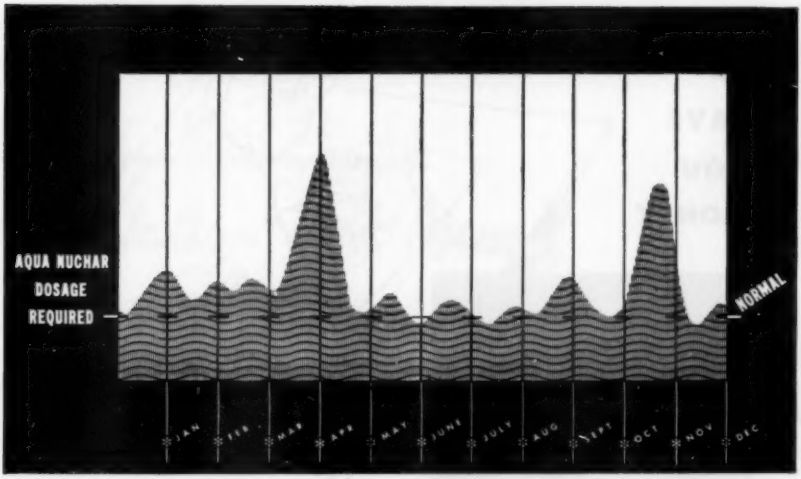
Section 2 of the specifications consists of a series of definitions, intended to establish a common terminology and avoid confusion. (In different sections of the country component parts are named differently.) Section 3 establishes a standard nomenclature by means of drawings of both oil- and water-lubricated pumps, with an accompanying table identifying the component parts. Section 4 includes a recommended general specification form, together with specifications for both oil- and water-lubricated deep well pumps and columns. Section 5 presents engineering data, including a friction loss chart for standard columns, a head loss chart and a mechanical friction chart for turbine-pump line shafts, and a table for line shaft size selection. Section 6 deals with factory inspection and tests, and includes an illustration of a typical laboratory test arrangement.

Although this tentative specification does not represent the ultimate in standards for vertical turbine pump use, it will assure those who use it of the reliability of their equipment, regardless of manufacture and should prove especially valuable to the small operator. Yet it is not so restrictive as to compromise the individual features of any manufacturer.

Grateful acknowledgment is made to the co-chairman, Perry H. Brown; to the secretary, Dan Rankin; and to Wilson L. Dornaus, Clark D. Bower, Thomas R. Mackay, Aladar Hollander, John L. Perhab, Fenmore Dunn, H. W. Iversen, and all the other committee members who have made this standard possible. If the members of the AWWA and the users of vertical deep well turbine pumps can make effective use of this standard, the efforts of the committee will indeed have been justified.

Reference

1. American Standard Specifications for Deep Well Vertical Turbine Pumps—ASA B58.1 (AWWA A101). Am. Wtr. Wks. Assn., New York (1955); *Jour. AWWA*, **47**:703 (Jul. 1955).



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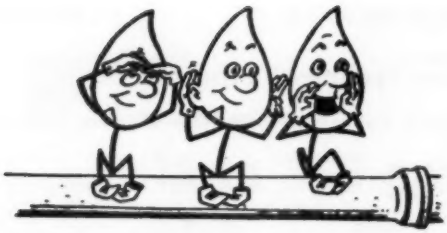

Percolation and Runoff

The St. Louis blues are beginning to set in now for the unlucky few who aren't going to be able to make AWWA's Diamond Jubilee Conference next May 6–10 back at the Association's Missouri birthplace. What they are going to miss is much more than an old home week, for, though the accent will be on venerability—with a session devoted to a review of the water works progress in AWWA's time, with certificates of honor presented to Associate Members whose affiliation with the Association dates back 50 yr or more, and with even the exhibits giving recognition to AWWA's new status as a three-quarter-centurian (septu-and-a-half-agenarian, that is)—most of the program (see pp. 36, 38 P&R) will be pointed at the future. More than a gloating over past accomplishments, this 76th annual meeting promises to be the beginning of the sprint through the last quarter of AWWA's first century. As a matter of fact, with membership pushing 11,000, with new and bigger headquarters, and with a larger JOURNAL under way, AWWA's outlook throughout is forward. If you're looking forward, too, plan to get to St. Louis early and to leave late; if you can't make it, don't let the blues get you—make the week of May 6 Water Works Week at home!

Water sports in Ohio are beginning to play things a little closer to the vest it seems. At any rate, where the water commissioners of Toledo and Cleveland, last summer, were offering $100 and $50, respectively, to any of their customers who could drink 2 cents worth of water in a week, the odds have now shortened to the point where water superintendent W. H. Shawhan, at Greenfield, is making his offer on the basis of $50 for drinking 5 cents worth in a week. The strange thing is, though his odds seem shorter, Shawhan is actually taking more of a risk, for his nickel's worth is 100 gal, where Toledo's 2 cents represented 116 gal and Cleveland's, a "sure thing" at 234 gal. Whatever the odds, the gimmick seems to be winning water the reputation of being a bargain—which can be the first step from water sport to water support.

An opening for a registered civil engineer with considerable experience exists in the Engineering Dept. of Muskegon, Mich. Those interested can obtain further information and application forms from T. J. Mangin Jr., Director, Board of Civil Service Commissioners, City Hall, Muskegon, Mich.

(Continued on page 40 P&R)

AWWA'S New Address: 2 Park Ave., New York 16, N.Y.

St. Louis Conference Technical Program

Open Session—Water Works Administration Committee—9:30 AM—Monday, May 7

Task Group Reports
- NARUC Rules and Regulations.....John H. Murdoch Jr.
- Rules and Regulations of the California Commission.....W. C. Welmon
- Job Classification.....Robert S. Millar
- Rating Water Systems.....John H. Murdoch Jr.

Survey of Mobile Radio Use, 1955.....M. B. Cunningham

Purification Division—9:30 AM—Monday, May 7

Calcite Stabilization of Lime-softened Water.....H. O. Hartung
- Discussion.....V. J. Calise

Panel Discussion—Water Treatment Section, State Sanitary Engineers Manual—Plans for Public Water Supplies.....*Led by* Philip Morgan
- Task Group Statement.....W. W. Aultman & E. H. Aldrich
- Discussion.....C. W. Klassen

Purification Division—2:00 PM—Monday, May 7

New Algicides.....C. M. Palmer

Economics of Sludge Removal.....Fred G. Gordon

Coliform Limits for Raw Waters.....Graham Walton

Diatomite Versus Conventional Filter Performance.....G. R. Bell

Joint Session—Resources and Management Divisions—2:00 PM—Monday, May 7

Panel Discussion—St. Louis Area Water Supply Developments.....*Led by* C. M. Roos
- The Water Resources of the Area.....C. M. Roos
- Water Resources Development on the East Side of the Mississippi River....S. C. Casteel
- Water Resources Development for the City of St. Louis.....John B. Dean
- Water Resources Development in St. Louis County.....W. V. Weir
- Water Quality and Treatment Requirements .....W. B. Schworm

Shall Recreational Uses of Water Works Impounding Reservoirs Be Permitted?.....
Robert B. Diemer, Merrill L. Riehl & Alexander Minkus

Joint Session—Management and Resources Divisions—9:30 AM—Tuesday, May 8

The Status of Federal Highway Legislation.....Harry E. Jordan

The Nation's Water Resources.....Fred G. Aandahl

Panel Discussion—The Missouri River.....*Led by* John W. Cramer
- Effect Upon the Total River Flow of Present and Probable Future Operation of Main-Stem Dams.....Wendell Johnson
- Recent Changes and Trends in Quantity and Quality of Missouri River Water and of Ground Water in the Immediate Vicinity of the River.....George E. Ferguson
- Present Conditions and Trends in the Pollution of the River.....Dwight F. Metzler
- Experiences and Observations of the Water Utility Executive.....
John C. Detweiler & James B. Ramsey

(*Continued on page 38 P&R*)

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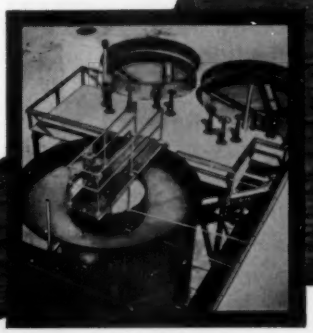

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(*Continued from page 36 P&R*)

Open Session—Water Works Practice Committee—2:00 PM—Tuesday, May 8

Progress Report—Meter Standards.....James G. Carns
Research Reports
Effect of Water Treatment Methods on Water Main Carrying Capacity.....T. E. Larson
Toxicity Studies on Cadmium and Chromium in Public Water Supplies.....C. F. Decker
General Policy Committee Report on Conformance to Standards.....M. B. Cunningham

Resources Division—2:00 PM—Tuesday, May 8

Water Demand Potential of Irrigation in Humid Areas.....John R. Davis
The Evaluation of Weather Modification Experiments.....S. A. Berry
Panel Discussion—Resources Section, State Sanitary Engineers Manual
Task Group Report.....Louis J. Alexander & Samuel B. Nelson
Discussion.....B. A. Poole

Distribution Division—9:30 AM—Wednesday, May 9

The Control of Booster Stations to Serve Secondary Areas.....Marvin H. Owen
Centralized Load Dispatching Experience.....J. M. Jester & J. W. Henderson
Problems in Purchase of Water—Demand or Penalty Rates.....V. C. Lischer
Surge Control on Transmission Lines of the St. Louis County Water Company.....
F. E. Dolson

Management Division—9:30 AM—Wednesday, May 9

Management Reorganization of Philadelphia Water Department.....Samuel Baxter
Modernizing the Metering Program of the Philadelphia Water Department.....
Gerald E. Arnold
How Meters Help Increase Revenue.....Wentworth Smith

Business Session—Management Division—2:30 PM—Wednesday, May 9

Discussion of State Sanitary Engineers Manual.....
Paul Haney, M. P. Hatcher & Dwight F. Metzler

Business Session—Distribution Division—2:30 PM—Wednesday, May 9

Discussion of State Sanitary Engineers Manual.....
Russell G. Kincaid, L. S. Finch & Earl Devendorf

General Session—10:00 AM—Thursday, May 10

Seventy-Five Years' Improvement in Water Supply Quality.....Abel Wolman
Seventy-Five Years' Progress in Materials and Construction.....E. Sherman Chase
Seventy-Five Years of Too Cheap Water.....John H. Murdoch Jr.

Distribution Division—9:30 AM—Thursday, May 10

Behavior of Steel Pipe Under Earth Loads.....R. E. Barnard
Application of Soil Mechanics to Pipeline Construction.....Henry Reitz
External-Corrosion Problems in the Water Distribution System.....L. P. Sudrabin

Management Division—2:00 PM—Thursday, May 10

Service Requirements of Water-connected Devices.....James G. Carns
Galvanic Corrosion in Water Meters.....H. F. Barrett
Studies in Water Use.....Ross Hanson & H. E. Hudson Jr.
The In-Service Training Program of the Philadelphia Suburban Water Company.....
Kenneth E. Shull & George H. Dann

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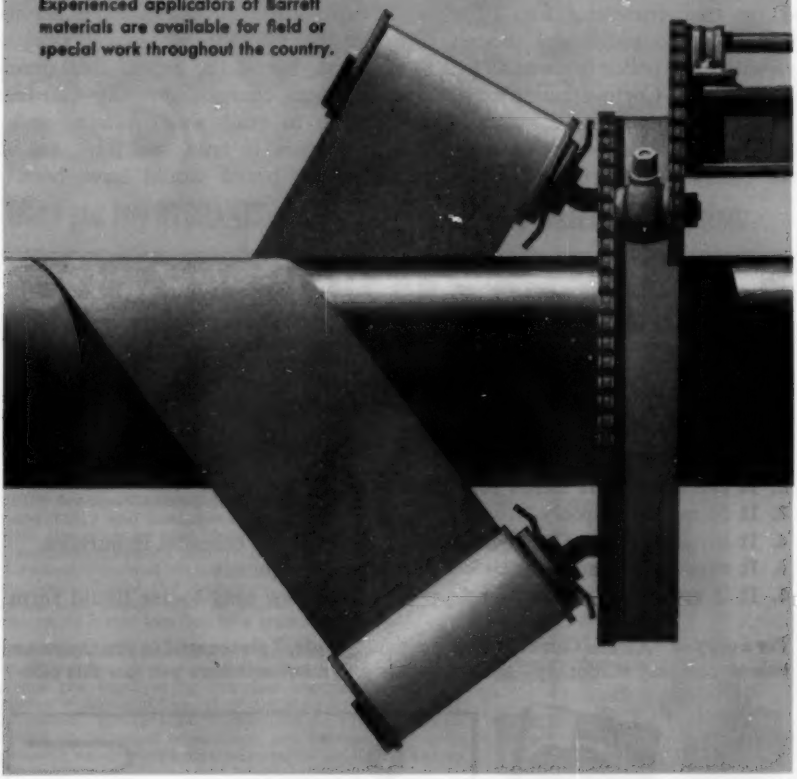

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(Continued from page 35 P&R)

Confu at Corfu—confusion at Corfu, N.Y., that is, with the entire water department staff, the full police force, and the rubbish collection and tree-trimming crew all off the job at once for a 10-day stretch, beginning last Mar. 1—suspended by the village board for "dereliction of duty." Not to question the wisdom of the village board, we can hardly help but wonder if they considered who would really take the beating. After all, though getting along without rubbish collection or tree trimming for 10 days wouldn't be too tough and though 10 days without a police force in as peaceful a town as Corfu shouldn't set off any major crime waves, 10 days without water—anywhere—might very well make voters thirsty enough and dirty enough and a good many other things enough to take the board to task. But that is merely idle speculation. Undoubtedly the board had in mind pitching in itself to keep the water flowing or, perhaps, spending the entire $90 that it saved on the suspension to hire temporary help to keep the system in operation.

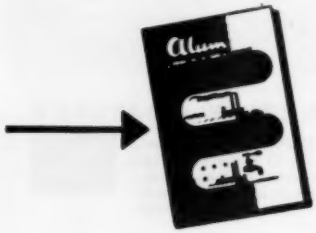
That $90 figure is no misprint, by the way; it represents the full total pay, at the $9 per day that Leonard J. Gardner, who is all these things to Corfu, earns for his multiple services. We don't know the inside story, of course, but if the report that dereliction was charged only in Gardner's failure to read water meters and to trim trees is true, we have an idea that the board would have been on

(Continued on page 42 P&R)

Depend on Cyanamid's ALUM for 6 good reasons

1. It feeds uniformly, without trouble, in solid or liquid form.
2. It has a wide pH range for effective coagulation.
3. It forms floc rapidly.
4. It gives maximum adsorption of suspended and colloidal impurities.
5. It causes minimum corrosion of feeding equipment.
6. It is available in granular form or in the new easy-to-use liquid form.

For a copy of "Alum—Commercial Aluminum Sulfate," please send us your name and title on company stationery—and would you also mention where you saw this offer?



AMERICAN Cyanamid COMPANY
Heavy Chemicals Department
30 Rockefeller Plaza, New York 20, N. Y.
In Canada: North American Cyanamid Limited, Toronto & Montreal

When you buy HYDRANTS and VALVES for your city waterworks system...

LOOK TO THE YEARS AHEAD, TOO!

EDDY can supply replacement parts for any Eddy Valve or Hydrant ever installed! This is important to you because the Valves and Hydrants you buy today must serve several generations yet unborn. You want no "orphans" in your system. Eddy Hydrants and Eddy Valves are backed by Eddy Valve Company's 104 years of dependable operation—your assurance that replacement parts will be available during the years to come.

EDDY Bronze-Mounted HYDRANTS are built for dependability and lasting service. They open smoothly *with* the pressure and close without water hammer. One man can easily remove all operating mechanism for inspection and repair. Positive drip action automatically drains the standpipe, safeguarding against freeze-ups. Stem held in place below main valve means no water loss due to a bent stem.

EDDY Bronze-Mounted GATE VALVES offer simplicity of design, trouble-free operation and enduring service. From the engineering drawing boards through all stages of manufacture, they are step by step a truly "finished" product of workmanship. These factors, added to personal experience, are reasons why progressive water works men have relied on Eddy Valves for years.

Eddy Hydrants and Valves are available with bell, flange or mechanical joint connections to fit any existing or planned installation.

Send today for full information on the complete line of Eddy Valves and Hydrants. No obligation.

EDDY VALVE COMPANY
A Subsidiary of Adams & Close & Sons
WATERFORD, NEW YORK

JUST LIKE MOTHER USED TO MAKE

Mom's home-made bread was mighty good . . . but so is today's store bread (and probably better for you and much cheaper, too). Ace works the same way . . . highly trained water main cleaning crews using the latest patented, exclusive equipment mean much lower cost plus maximum pipe efficiency for you. Easier, faster, more efficient results . . . and results count. And you can count on Ace for water mains cleaned . . . with up to 95% of efficiency restored (at surprisingly low cost). Investigate now.

A CALL IS ALL...

Call collect, Chestnut 1–2891, Kansas City, Missouri. More facts, free estimates and surveys are yours for the asking. No obligation.

Ace PIPE CLEANING, INC.

General Offices
3513 East 18th Street
Kansas City, Missouri

(*Continued from page 40 P&R*)

sounder ground just semisuspending him—perhaps $21.38 worth, or some other equitably determined amount. Actually, though, it isn't exactly easy to see how any suspension would help get meters read or trees trimmed, but that probably involves some special insight into penal psychology.

There are Corfus all over the place, of course—many of them with fewer than Corfu's 500 population and many, too, with water department payrolls smaller than the approximately $150 per month that Gardner's water department services must cost. It's the "confu" that made this one rate Associated Press coverage and that may, just possibly, lead to augmentation of Corfu's public employee staff. It's an ill thing, though, that it takes an ill wind, and that maybe even that may do no more than blow 90 extra dollars into Corfusian coffers. In that event, just plain "fu!"

That homebodies are one of the principal sources of increasing water demand is an obvious conclusion from recent statistics compiled by F. W. Dodge Corp., New York. The Dodge report points out that contract awards for the construction of single-family homes in the 37 eastern states have almost doubled (up 97 per cent) since 1951, with a gain in 1955 of 21 per cent over 1954, the previous record year. The extent of growth of one-family housing tends to be obscured in the figures for total residential building, which rose only 64 per cent during the 4-year period, because multiple-dwelling construction has been declining. The expectation is that home building will continue at a high level for the next decade. And they all have lawns!

(*Continued on page 44 P&R*)

CITY OF SAN FRANCISCO 44" RIVETED STEEL LINE INSTALLED IN 1888. THE LINE IS STILL IN SERVICE.

OLD AGE SECURITY
WITH STRONG STEEL PIPE

Dependable service through the years is yours when you choose durable steel pipe for your water system. As evidence we could cite case after case where steel pipe lines have given 70, 80 or even 90 years of trouble-free use. And that is only the beginning of a long life story. With today's technological improvements in steel pipe you can now count on even *longer* years of *more* dependable service.

In every way reliable service becomes an old, old story . . . when you use stronger steel pipe. Tough resilient steel—made to last—assures you a safe and secure pipe that will best serve your needs today—and for 100 years to come.

So if you want "old age security" in your water line, you're smart to specify STEEL pipe.

"WHEREVER WATER FLOWS—STEEL PIPES IT BEST"

STEEL PLATE FABRICATORS ASSOCIATION

79 W. MONROE ST., CHICAGO 3, ILL.

Fordin' Along...

"A daily need for every living thing. It rises from the earth obedient to the summons of the sun, and descends in showers of blessings. It gives of its beauty to the fragrant flowers; it is the alchemy that transmutes base clay to golden grain. It is the canvas on which the finger of the infinite traces the radiant bow of promise. It is the drink that cheers, and brings no sorrow. Jehovah, upon Creation's dawn said, 'It is good'." William Jennings Bryan once gave this brilliant toast to water at a dinner in his honor.

(*Continued from page 42 P&R*)

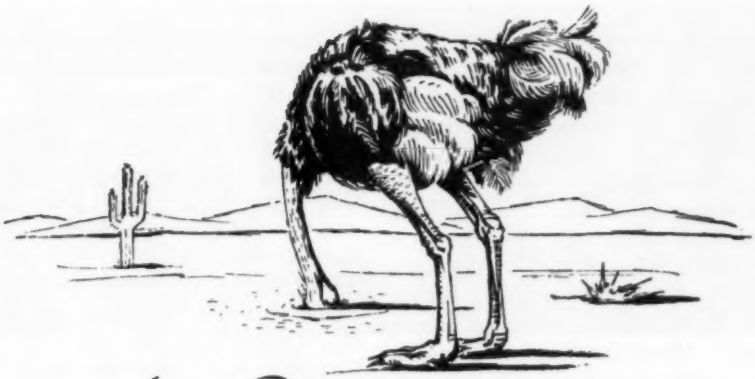
William C. Sherwood has retired after 26 years as secretary of Hersey Manufacturing Co., South Boston,

Mass. Starting as a clerk with the company's New York office in 1900, he eventually became manager there. In 1927 he moved to the main office in Boston as assistant to the president, later serving as sales manager. He was named secretary in 1930. Mr. Sherwood was four times president of WSWMA, and has twice represented that organization as AWWA director.

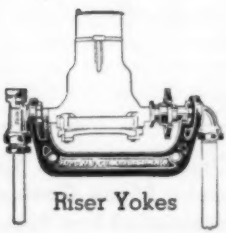
Oil and water mix in the case of Ross L. Dobbin, retired general manager of the Peterborough (Ont.) Utilities Commission. The former AWWA president was presented with his portrait in oil on Jan. 9, Ross Dobbin Day at the Peterborough Rotary Club. The presentation was made by W. R. Strike, vice-chairman of the Ontario Hydro-Electric Power Commission, which named a substation in his honor.

G. A. McBride, midwestern district sales engineer for Infilco Inc., has been named vice-president of the Michigan Sewage & Industrial Waste Assn.

(*Continued on page 46 P&R*)



An Ostrich is an EXPENSIVE METER MAN



Riser Yokes

Angle Yokes

Straight Line Yokes

And so is anyone else who must stand on his head to set or change a meter. Old fashioned, hard-to-get-at connections, outdoors or in the basement, are time consuming and costly. Not only is the meter hard to change, but periodic efficiency checks are neglected and water revenues are lost due to inaccurate meters.

Be sure your system is modern. See that Ford Meter-setting Yokes are keeping every meter available for quick changing and testing. Savings in time, and increased water revenues are often enormous.

You can change a meter in a Ford Yoke

EASILY

There are models for every condition. Send for free catalog.

FORD

FOR BETTER WATER SERVICES

THE FORD METER BOX COMPANY, INC. Wabash, Indiana

(*Continued from page 44 P&R*)

Treeson, if not actually treason, is what we have to call the action of one of the water experts who addressed the "Water for Texas Conference" at Texas A&M last fall. Asked the best way to water trees, his worse than subversive suggestion was: "put the hose at the base of the tree and turn on a tiny trickle. Let it run continuously —water meters don't measure such a small flow and it won't cost you a penny." Were the "expert" anything but anonymous and were Texas anything like it used to be, we would certainly expect one of the supple limbs of one of those well-watered trees to be supporting the weight of an exexpert.

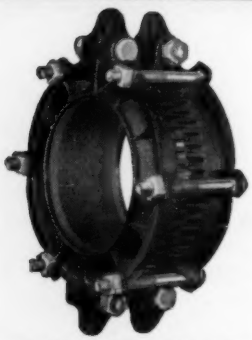
The "ivory tower" aspects of university life may have been well illustrated by a recent report in *Chemical & Engineering News* on a new modification of boron monoxide prepared at Pennsylvania State University. According to the report, "it was prepared when sub-boric acid, $B_2(OH)_4$, was converted by dehydration into the *hitherto unreported form,* BO [italics ours]." Where, we wonder, have they been all these years?

Not at the University of Pennsylvania, certainly, for there the tower has seemed more lifebuoy than ivory ever since Cornellians pinned on their archrivals the not "best friendly" abbreviation, PU!

Arthur A. Baker has been appointed associate director of the US Geological Survey. He joined the agency in 1921 and has been serving as administrative geologist in the office of the director since 1953.

(*Continued on page 48 P&R*)

FOR REPAIRING BROKEN MAINS



Skinner Seal Split Coupling Clamp. One man can install in 5 to 15 minutes. Gasket sealed by Monel band. Tested to 800 lbs. line pressure. A lasting repair. 2"-24" inclusive. Write today for new catalog.

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SOUTH BEND 21, INDIANA, U.S.A.

For Organic Iron and Color Removal—Algae Control—Regeneration of Iron Removal Filters—

Use

POTASSIUM PERMANGANATE

Supplied to industry and municipalities in various grades and meshes to fit the particular operation efficiently and economically in combating many water problems.

WRITE FOR QUOTATIONS

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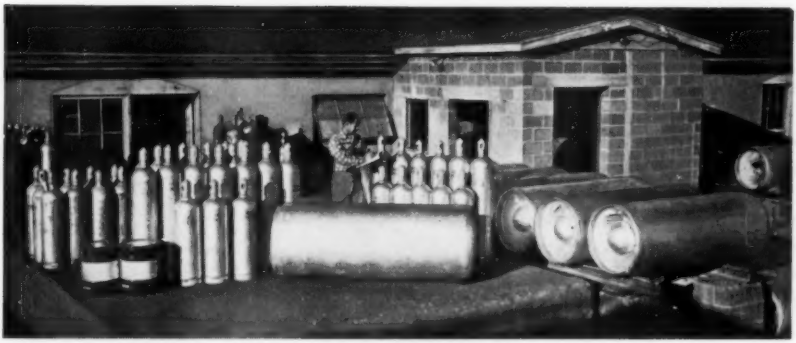

AT EACH of the six Jones Company plants, Chlorine is received in tank cars, repacked in smaller containers selected to meet the customer's needs.

Save storage space and shipping time on your next CHLORINE purchase!

Get fast delivery on *many* chlorine products in whatever container best fits your needs.

Stop the cost and nuisance of storing huge amounts of Chlorine. Buy only what you need—whether it's several carloads, just part of one tank car, or as little as 16 lbs.!

You can get less-than-carload shipments of Chlorine from John Wiley Jones. And you get them quickly, because Jones has six distribution centers spotted in important locations around the country.

Eliminates searching

You save searching time, too, because you have a *single* convenient and dependable source for *all* your Chlorine products: liquid Chlorine. Calcium Hypochlorite, and Sodium Hypochlorite.

Cylinders, tins, drums, carboys, and ton-tanks—Jones delivers in any type of container you require. This gives you complete protection against sudden emergency needs for Chlorine.

Top quality

Jones Chlorine is quality controlled to meet high government standards. Municipalities, public utilities and industrial users have relied on Jones Chlorine for a quarter of a century.

Dependable

Our trained technical staff will be glad to help you solve *your* Chlorine problem. Write for prices.

LIQUID CHLORINE
In 16-, 105-, 150-lb. cylinders and 1-ton tanks.

CALCIUM HYPOCHLORITE
(70% available Chlorine) In 3¾- and 5-lb. tins, 100-lb. drums.

SODIUM HYPOCHLORITE
In gallons, carboys. Tankwagon deliveries in 1000-3000 gal. lots.

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or Dial 81-6-6288

(*Continued from page 46 P&R*)

Saline water conversion projects and research were the subject of a progress report issued in January by the Office of Saline Water of the US Dept. of the Interior. It was stated that the most promising processes so far developed appear able to produce fresh water "at costs comparable with the maximum present costs of industrial water supplies." The report cautioned, however, that such maximum costs are exceptional and that "any extensive use of freshened saline water is dependent on reducing present production costs." This reduction will require a number of years of intensive effort, including the construction and operation of pilot plants. With regard to various conversion processes, the report said:

1. Distillation shows much promise of attaining considerably lower cost levels. It is anticipated that expansion of development work to fullest utilization of very high heat transfer levels will be productive. Combination processes within the normal temperature range of distillation, as well as work in the critical range of salt water, also require further study. Scaling has proved to be a formidable difficulty. For research into both distillation and scaling control, experimental work on actual sea water at a seashore location is essential.

2. The performance of membranes, both electric and osmotic, is noteworthy and justifies continuing support. The investigations of the past 2 years reveal the possibility that fulier work in both laboratory and pilot plant research will result in greatly improved performance and lower water costs.

(*Continued on page 52 P&R*)

ELEVATED TANKS

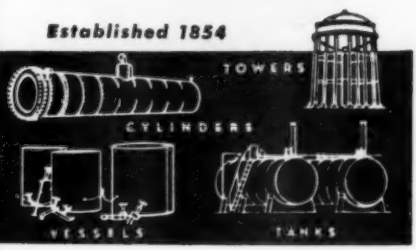
For almost a century Cole elevated tanks have been helping provide uniform water pressure, fire protection and adequate water reserve in scores of American cities.

Capacities 5,000 to 2,000,000 gallons—with hemispherical, ellipsoidal or conical bottoms. Also flat-bottom tanks for standpipe storage. Correctly built in accordance with AWWA specifications.

We invite your inquiries. State capacity, height to bottom, and location.

R.D. COLE
MANUFACTURING CO.
NEWNAN, GA.

Established 1854



DELIVERING WATER CHEAPER

Engineered and built by the city of Syracuse, N. Y., this 38.7″ Dresser-Coupled steel water supply line was backfilled immediately after each 30′ length was joined, getting streets back to work fast, practically eliminating the problem of trench cave-ins.

Too Quick for Cave-ins!

Dresser-Coupled steel line permits backfilling without delay

Whenever conditions demand immediate backfill, steel pipe and Dresser Couplings make it surer, safer to comply.

On this Syracuse water supply job, for example, the cave-in hazard of unstable sandy loam was heightened by heavy traffic nearby. The problem was solved by immediate backfilling, a standard Syracuse practice on all Dresser-Coupled water lines, where leaving trenches open for line test during construction is considered unnecessary.

Proved in thousands of installations, these factory-tested couplings give 100% bottle-tight joints every time. Their resilient rubber gaskets absorb vibration and pipe-transmitted stresses . . . assure no-leak safety for the life of the line.

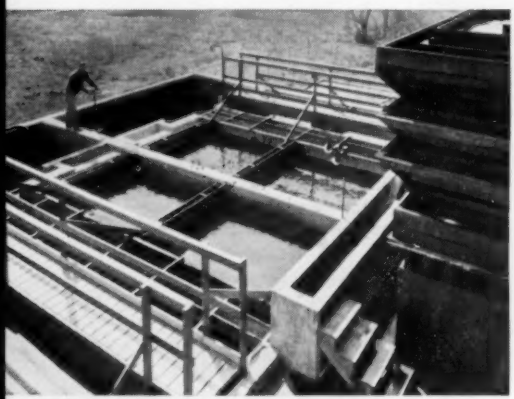
Dresser Couplings and steel pipe provide trouble-free lines that always *deliver water cheaper.*

Wherever water flows, steel pipes it best. Always put steel pipe and Dresser Couplings in your specifications. Dresser Manufacturing Division, Bradford, Pa. Sales offices in: New York, Philadelphia, Chicago, South San Francisco, Houston, Denver, Toronto, Calgary.

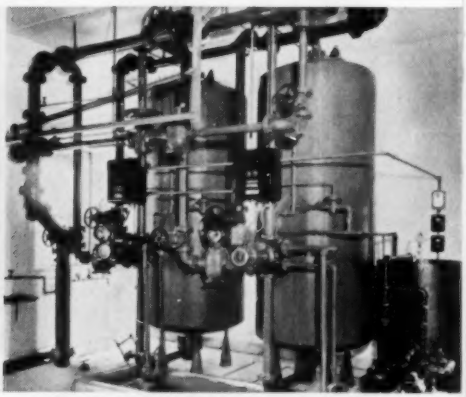
ONE OF THE DRESSER INDUSTRIES

TURBIDITY gets cut from 1500 to less than 5 ppm in this Precipitator. Mt. Carmel, Ill. Cons. Eng.: *Warren & Van Praag, Inc.*

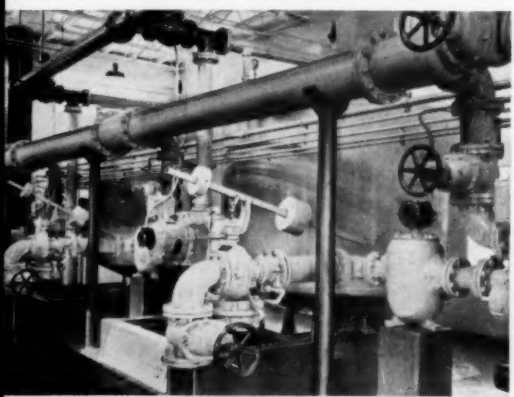
SEA WATER saves $32,980 per year over salt for regenerating Softeners. Hollywood, Fla. Cons. Eng.: *Reynolds, Smith & Hills.*



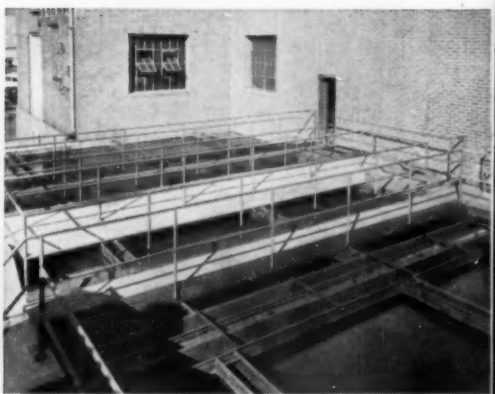
30 PPM IRON gets cut to 0.14 ppm in Aerator and Precipitator . . . even before filtration! Fairless Hills, Pa. Cons. Eng.: *H. H. Le Van, Jr.*



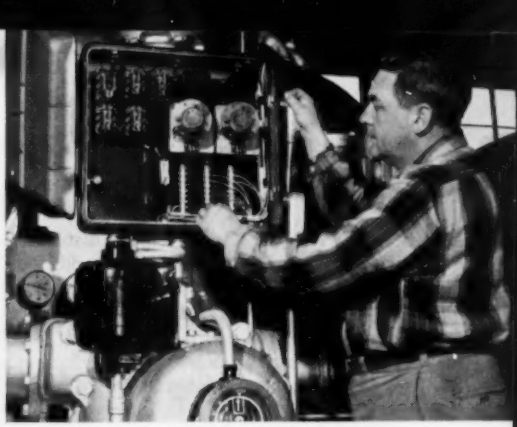
269 PPM HARDNESS is removed by these automatic Softeners. Aerator and Filters cut iron. Martinsville, Ill. Supt.: *O. Wiser.*



AUTOMATIC CONTROLS backwash, regenerate, rinse and return Softeners to service. New Smyrna Beach, Fla. Cons. Eng.: *Russell & Axon.*



OLD SETTLING BASINS house these modern Precipitators that boost capacity. North Kansas City, Mo. Cons. Eng.: *C. A. Haskins.*



HARDNESS TESTER (left) initiates regeneration by Automatic Controls (right). This new Permutit plant, with Q resin, softens 72% more water—using same manpower, less salt! E. Aurora, N. Y. Village Eng.: *E. J. Maurer.*

Why these cities chose PERMUTIT® *water conditioning*

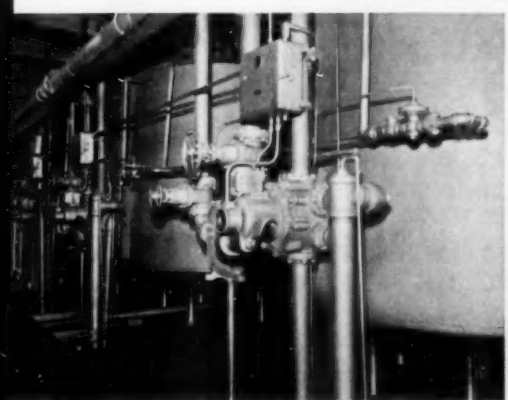
Some chose Permutit for its world-known engineering experience gained through thousands of installations covering every major development in clarification, filtration and softening equipment.

Others chose Permutit to get complete, integrated service: the water-analysis labs, engineering, equipment, controls, ion exchange resins—all from one source, under one responsibility.

But most of the designers of these plants chose Permutit because they have personal knowledge of many installations where Permutit equipment has given long years of efficient, trouble-free performance. They know that's what pays off in the long run.

Write for Technical Bulletins. The Permutit Company, Dept. JA-4, 330 West 42nd St., New York 36, N. Y.

SOFTENERS • ZEOLITES and RESINS • AUTOMATIC CONTROLS • GRAVITY FILTERS • PRESSURE FILTERS
PRECIPITATORS • CHEMICAL FEEDERS • AERATORS • SWIMMING POOL SYSTEMS • IRON REMOVAL EQUIPMENT



SOFTENER CAPACITY TRIPLED and $7,000 yearly saved on salt by change from zeolite to Permutit Q. Normal, Ill. Cons. Eng.: *Farnsworth & Conley.*

TRIPLE TROUBLE! 275 ppm hardness, 200 ppm alkalinity, 3 ppm iron are reduced by this Precipitator. Staples, Minnesota. Supt.: *E. H. Klang.*

(Continued from page 48 P&R)

3. Developments in solar distillation are encouraging and warrant additional work to bring this means of utilization of low-cost energy nearer to practical serviceability.

4. Freezing processes have inherently low cost if a rapid and economical method of separating the pure ice from the mother liquor can be developed.

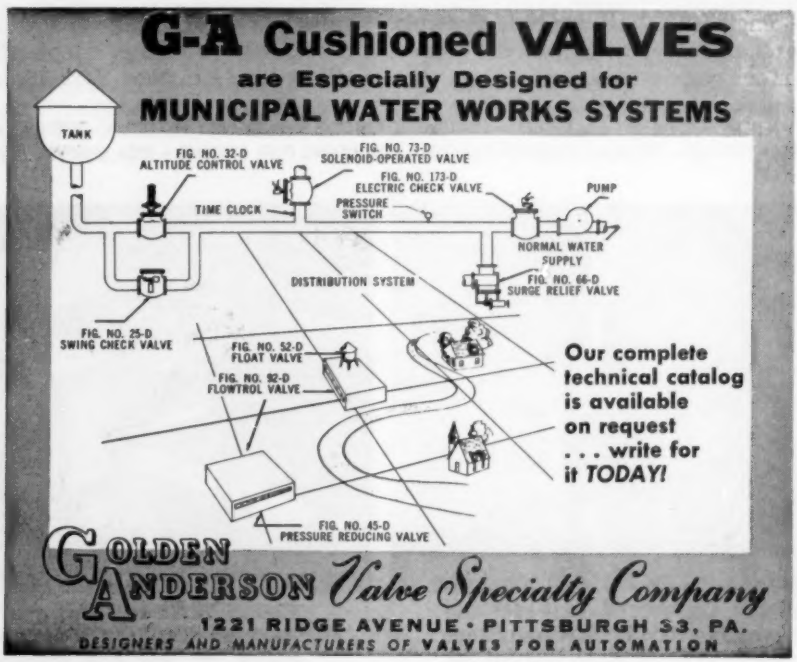
In addition, it is apparent that the several processes not yet fully explored, including chemical, physical, and electric, need to be followed to definite conclusions.

Weld inspection by radiography is said to be facilitated by a new line of X-ray equipment offered by Balteau Electric Corp., Stamford, Conn. The "Baltospot," intended for spot-checking 1½-in. steel or equivalent, weighs 46 lb and provides a power penetration of 140 kvp. The entire unit fits into a 32 × 16 × 13½-in. trunk.

Dorr-Oliver Inc., Stamford, Conn., has integrated its Canadian operations with those of E. Long Ltd., of Orillia, Ont. The firms have long been associated. The president of the new Canadian organization, known as Dorr-Oliver-Long Ltd., is Clarence R. Long, a director of Dorr-Oliver and president of E. Long. Headquarters are at Orillia, with the Toronto office of Dorr-Oliver and the Vancouver office of E. Long operating as divisional sales offices.

Plastic pipe and fittings will be marketed by a new division of Smith-Scott Co., Riverside, Calif. The company has signed an exclusive sales agent agreement with Kerr Industries for the latter's entire factory production of rigid polyvinyl chloride pipe.

(Continued on page 54 P&R)


G-A Cushioned VALVES
are Especially Designed for
MUNICIPAL WATER WORKS SYSTEMS
TANK
FIG. NO. 32-D ALTITUDE CONTROL VALVE
FIG. NO. 73-D SOLENOID-OPERATED VALVE
FIG. NO. 173-D ELECTRIC CHECK VALVE
PUMP
TIME CLOCK
PRESSURE SWITCH
NORMAL WATER SUPPLY
DISTRIBUTION SYSTEM
FIG. NO. 66-D SURGE RELIEF VALVE
FIG. NO. 25-D SWING CHECK VALVE
FIG. NO. 52-D FLOAT VALVE
FIG. NO. 92-D FLOWTROL VALVE
Our complete technical catalog is available on request . . . write for it TODAY!
FIG. NO. 45-D PRESSURE REDUCING VALVE
GOLDEN ANDERSON Valve Specialty Company
1221 RIDGE AVENUE • PITTSBURGH 33, PA.
DESIGNERS AND MANUFACTURERS OF VALVES FOR AUTOMATION

WORTHINGTON-GAMON

WATCH DOG WATER METERS

The meter used by thousands of municipalities in the U. S.

"Watch Dog" models . . . made in standard capacities from 20 g.p.m. up: frost-proof and split case in household sizes. Disc, turbine, or compound type.

SURE TO MEET YOUR SPECIFICATIONS FOR ACCURACY, LOW MAINTENANCE, LONG LIFE.

Before you invest in water meters, get acquainted with the design and performance advantages which make Worthington-Gamon Watch Dog Water Meters first choice of so many municipalities and private water companies in the United States.

WORTHINGTON-GAMON METER DIVISION

Worthington Corporation

296 SOUTH STREET, NEWARK 5, NEW JERSEY

OFFICES IN ALL PRINCIPAL CITIES

(Continued from page 52 P&R)

The hydrant, like the water works itself, too often rates the spotlight only with its worst foot forward, being noticed most when it gets in the way of a speeding car or takes up the only parking space in sight. And from the size of a year's stack of newspaper clippings on the subject, it would appear to do these dastardly deeds alarmingly often. Among last year's tales of death, disablement, and discomfort, of course, there were the accepted classics: the portable hydrant—this time in front of Moriarty's New York chop house to keep parkers from blocking the entrance to the establishment—and the fire hydrant afire—this one at Bridgeport, Conn., from gas in the pipes—as well as a few new wrinkles. Best of these, no doubt, was the story from La Jolla, Calif., and Portland, Ore., where a dentist and a beautician, respectively, obtained permission from their water departments to chrome-plate the hydrants in front of their places of business, thus to counteract their plug-ugliness. And worst had to be the news from Hicksville, N.Y., noting that the annual Labor Day Fire Tournament, which brought teams from 50 Long Island fire departments and 50,000 spectators to town, was cut from seven events to two because a leak in the hydrant on the tournament grounds reduced pressure to practically zero. Not likely, with a press such as this, to be recognized as man's best friend, the hydrant must apparently be content with a role as man's best friend's best friend.

(Continued on page 80 P&R)

THERE'S NO LEAKAGE....
NO LUBRICATION
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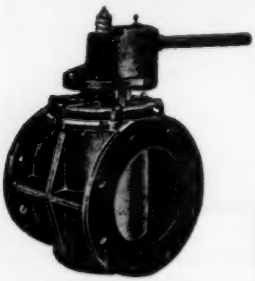

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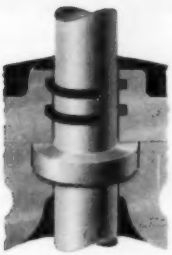

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Fig. 571

Fig. 561

Fig. 566

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Key: In the reference to the publication in which the abstracted article appears, **39**:473 (May '47) indicates volume 39, page 473, issue dated May 1947. If the publication is paged by the issue, **39**:5:1 (May '47) indicates volume 39, number 5, page 1, issue dated May 1947. Abbreviations following an abstract indicate that it was taken, by permission, from one of the following periodicals: *BH—Bulletin of Hygiene (Great Britain); CA—Chemical Abstracts; Corr.—Corrosion; IM—Institute of Metals (Great Britain); PHEA—Public Health Engineering Abstracts; SIW—Sewage and Industrial Wastes; WPA—Water Pollution Abstracts (Great Britain).*

WELLS AND GROUND WATER

The Theory of Continued Ion and Concentration Balance in Subsurface Waters. J. JANAK. Prace Ustavu Naftovy Vyzkum, Publ. (Czech.), No. 4–8, p. 5 ('55). Geochem. study of deep subsurface waters in Czechoslovakian part of inner Alps Viennese Basin was undertaken. Theory was developed, interpreting origin and occurrence of various ions in subterranean waters. It was postulated that const. ion exchange exists among various underground regions and surface waters influencing chem. compn. of these waters. Gradient expressing numerically the difference in chem. compn. was formed as $K_{Cl}^{CO_3} = x$ and value for concn. differential as $y = F_{(x)}$, where y is depth in m of mineral salts deposit from given basin and F is function. It was assumed that salt concn. is directly proportional to depth from which this water originated. Surface waters contain appreciable SO_4^{--} and HCO_3^-, while underground waters near crude oil are rich in halogen ions, owing to leaching effect on prehistoric oceanic floors, rich in NaCl. Mineral content and chem. compn. are not always interdependent functions of geol. age sediment; steepness of ion concn. and chem. compn. slope is dependent on subsurface and aboveground mineral matrix, and finally secondary influences also have important roles in water compn. of genetic, tectonic, phys., and chem. nature.—*CA*

Determination of Speed of Filtration of Ground Waters by Method of Marked (Radioactive Atoms. V. BRESLAVS, G. MELIKA-SACHNAZAROVA & I. TAKSARS. Latvijas PSR Zinatnu Akad. Vestis (USSR), No. 3, p. 99 ('55). Lab. tests showed that $Co(NO_3)_2$ with Co^{60}, Na phosphate with P^{32}, and Na_2SO_4 with S^{35}, as well as several dyes, were all strongly adsorbed on silicates. Loss reached 80–90% in filtration of solns. through bed 5 cm thick. Thus, these compds. were not suitable for tracing movement of water in ground.—*CA*

Ground Water in the Delaware River Valley. H. C. BARKSDALE & S. M. LANG. Proc. Am. Soc. Civil Engrs., **81**, Separate No. 593 (Jan. '55). Yield of well in consolidated rocks of upper valley is dependent on size and no. of fractures encountered in drilling well. Small-to-moderate ground water supplies may be obtained from aquifers in this province, but great care should be taken in spacing of wells to reduce interference. Water obtained from aquifers of this area is generally fairly hard and moderately high in dissolved solids. It is, nevertheless, satisfactory for most purposes, frequently

(*Continued on page 64 P&R*)

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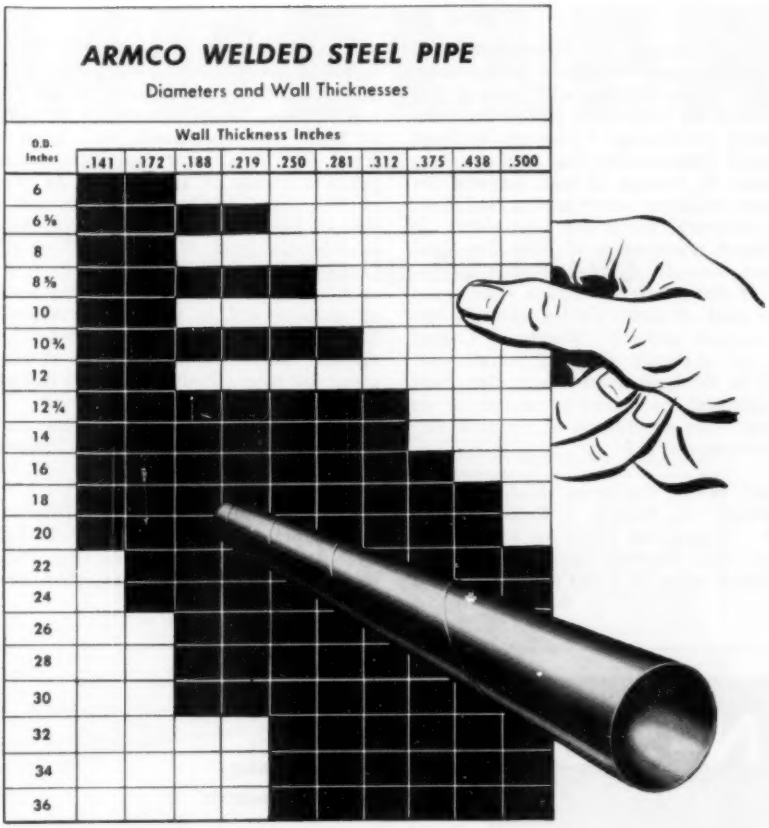

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(*Continued from page 62 P&R*)

without any treatment. Moderate-to-very-large supplies may be obtained from aquifers in Coastal Plain formations. Probably, best opportunity for large-scale ground water development in Delaware Valley can be found in Coastal Plain aquifers that crop out along Delaware R. because of high potential for recharge wherever water-bearing sediments are in hydraulic connection with river. In this respect, maintenance of satisfactory qual. of river water is of utmost importance because of direct influence which river water has on qual. of water yielded by wells tapping exposed aquifers. Waters of Coastal Plain are generally of good overall qual. except in localized areas where they have been contamd. by industrial wastes or by recharge from mineralized or contamd. surface waters.—*PHEA*

Ground Water Pollution With Gasoline at Wesel. K. FRICKE & KRAUSE-WICHMANN. Gesundheits Ingenieur (Germany), **74**:394 ('53). Several cases are described. In Munich, poln. by 1,000 gal lasted 2½ yr and extended up to 2 mi downstream with width of about 2,000'. Elsewhere, 180,000 gal of gasoline caused poln. lasting 4½ yr and reaching 1.6 mi downstream with width of 1,800'. At Andernach, well water was found pold. 25 yr after Americans had used gasoline storage at an airfield. At Wesel, in 1945, Americans had built large tank station in forest at edge of town and wasted considerable quant. of gasoline. First effect was killing of trees. In few months to 2 yr, neighboring wells became pold. independent of distance and in irregular arrangement. By 1952, poln. was found in wells 2,300' distant. Movement of poln. was, therefore, about 330 ft/yr. Pollution did not extend as far, but lasted longer, as at Munich. This fact is explained by backward flow during high waters in nearby Rhein R. Disappearance is said to be due to oxidation.—*M. Suter*

Hydrochemical Studies in the District of Düsseldorf. H. ARNOLD. Geol. Jahrb. Geol. Landesanstalt Bundesrep. Deut. (Ger.), **69**:481 ('55). Water samples, taken weekly

(*Continued on page 66 P&R*)

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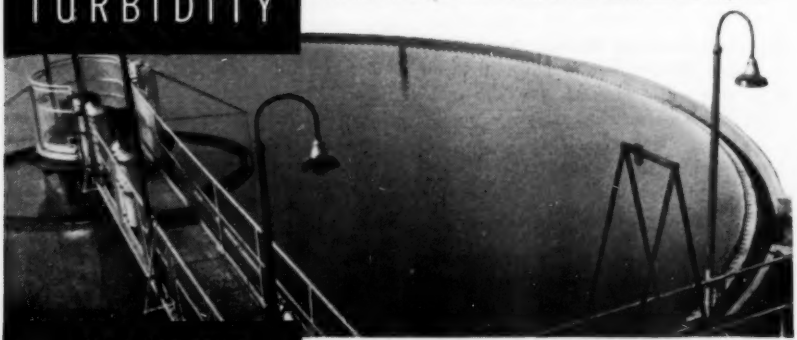

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(*Continued from page 64 P&R*)

from 20 different wells near Düsseldorf, varied during year in pH between 7.0 and 7.4, in total hardness between 16 and 28°, and in carbonate hardness due to carbonates between 8 and 17°; they contained 30–90 mg Cl/l and 0–1 mg Fe/l.—*CA*

Records of Selected Wells on the Eastern Shore Peninsula, Virginia. A. SINNOTT & G. C. TIBBITTS JR. Virginia Div. Geol. Mineral Resources Circ., **3**:39 ('55). Well logs and chem. analyses of ground water of 130 selected wells are presented in tabular form. Most wells derive water from sands or gravels, though some water-bearing beds consist of shells and shell fragments.—*CA*

The Relation of the Relative Ion Compositions of Waters of Several Bonds to Mineralization. M. N. TARASOV. Gidrokhim. Materialy (Russ.), **22**:79 ('54). In order to evaluate mineralization of waters in ponds, T. studies relations: (*a*) between content of prevailing anion *C* and sum of mineral substances $\Sigma\mu$, i.e. $\Sigma\mu = f(C)$; (*b*) between sum of mineral substances and relative ionic compn., i.e. % mg-equiv. $= f'(\Sigma\mu)$. These relations, established for one of ponds for no. of yrs, also hold true for 50 ponds in the area. By analytical detn. of SO_4 and with relations given above, approx. ionic compn. and total salts in waters of ponds can be calcd.—*CA*

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Rapid and Slow Filtration of Water of Lake Constance. The Waterworks of St. Gallen. E. HOFFMANN. Schweiz. Ver. Gas Wasserfach Monats-Bull. (Ger.), **34**:77 ('54). Rapid filtration takes place over 8 double beds of 40 sq m each, 1.0 m thick, 5 m per hr filtration rate, capacity 40,000 cu m per day. Part of it is slowly filtered through a bed 0.4–0.9 m thick, 5 m per day rate, capacity 15,000 cu m per day, part is chemically treated after rapid filtration. Numerous tabulations are given on sand sizing, amt. of microörganisms of various kinds and their removal, statistics on life of beds, backwash, etc.—*CA*

(*Continued on page 68 P&R*)

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No. 01 Cuts Pipe 4" to 8"
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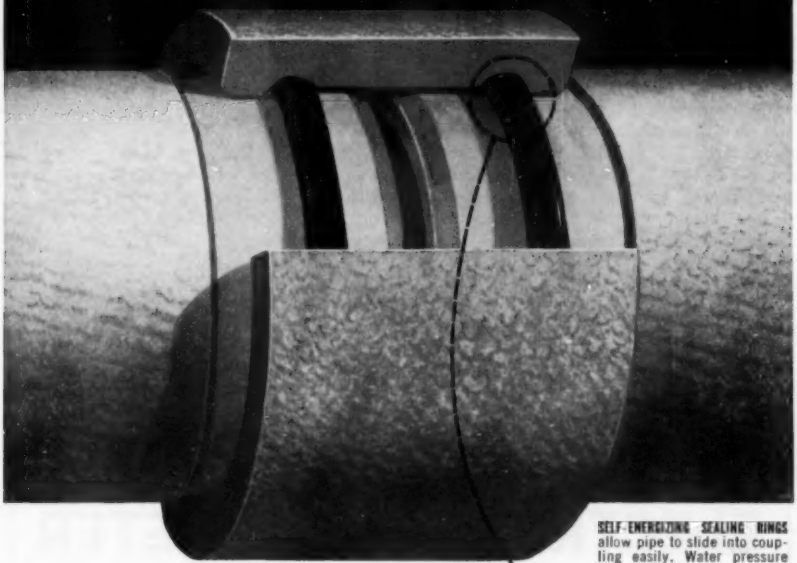

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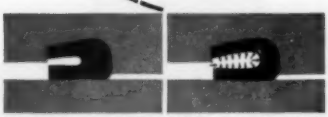

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K-M best in asbestos®

(*Continued from page 66 P&R*)

Using Wash Water Pumps as Meters for Measuring Rate of Rise of Wash Water in Rapid Sand Filters. R. W. ANGUS. Munic. Util. Mag. (Can.), 92:32 ('54). Frequently, low-head centrifugal pumps more convenient than elev. tanks for wash water. Occupy little space, first cost usually lower, and head const. Most common instruments for measuring wash water Venturi meter and orifice, both of which must be set in long, straight pipe to produce favorable approach conditions and give reasonable accuracy, which is not always convenient. Low-head, large-vol., high-specific-speed pumps, which have rel. steep head-discharge curves, may be used as meters with sufficient accuracy by use of differential gauge to indicate net head in terms of in. rise per min. Such gauge and method of connecting is described. Device reliable as long as pump in good condition. Not applicable to pumps with flat head-discharge curve over range of use.—*R. E. Thompson*

A New Treatment for Water Containing Organic Colloids. O. L. BIHET & H. GOLDSTEIN. Bull. mens. Centre belge Ét Document. Eaux (Belg.), 4:212 ('53). Experiments have shown that colloidal organic matter combined with iron salts can be removed from water by passage of water through filings of grey cast iron. It appears that electric currents set up by couples formed between iron and particles of graphite in it are sufficient to break up complexes. Plant for treatment of water by this process comprises filter containing iron filings (250–300 kg per cu m of water per hr) and sand filter. In tests of plant, water was applied at rate of 5 cu m per hr. Treatment of water from river Meuse to which additional organic matter was added, reduced concentration of iron in water from 1.5 mg to 0.1 mg per l. and permanganate oxygen demand from 55–65 mg to 10–20 mg per l., and removed all color.—*WPA*

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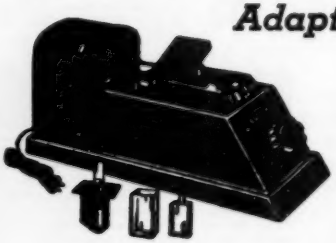

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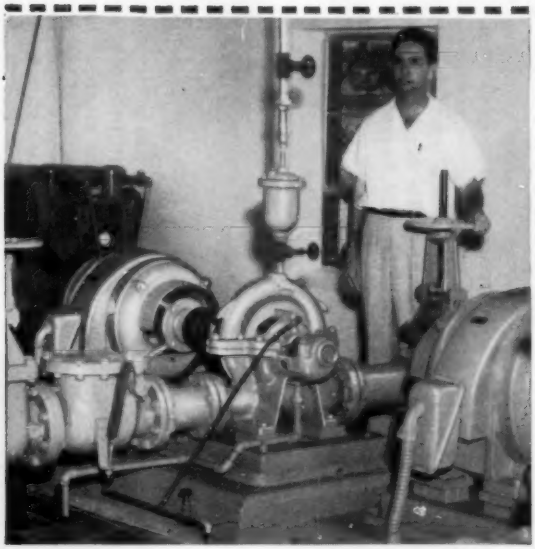

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A Savings of $30.00 a Month		

● Now, with modern Allis-Chalmers pumping equipment, it is possible for you to replace old outmoded pumps with new equipment — and pay for it out of power savings in an average depreciation period.

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Allis-Chalmers units are built with that extra measure of strength and ruggedness that will assure long life and low-cost service. Allis-Chalmers pumps and motors also save dollars in maintenance and downtime reduction.

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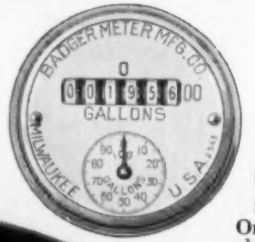
Metering your water supply is a step made easy by the Badger Meter representative in your area. He can furnish complete data on many types of Badger Meters best suited for the specific needs of your community. For an appointment, at your convenience, write direct.



Fair rates for all users - greater income per year

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Durability, dependability and low maintenance costs make community ownership of Badger Meters an investment in economy that pays dividends over the years.

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Water is Vital

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Legal Notes

This column devoted to court decisions and legislation of interest to the water works field will appear in the Journal from time to time. Readers are urged to send in reports and local press clippings which contain pertinent material.

Off-duty work—Wisconsin. The State Supreme Court upheld a Milwaukee Fire Dept. regulation prohibiting firemen from taking other jobs on their days off. The court stated: "The right to work for the public is a privilege which may be granted on any conditions which the public agency may impose, consistent with the law and public policy, and when an individual enters such employment, he impliedly surrenders certain natural rights which would remain his if he were a private citizen."—*US Municipal News, Jan. 15, 1956.*

'Advance' sewer rental—Ohio. On Jan. 10, 1956, the Fourth District Court upheld a lower court ruling that blocked collection of Gallipolis' sewer rental charge, an advance assessment to help pay for a new sewage plant. After the voters had turned down a proposed bond issue in 1954, the city began collecting the rental to finance a plant for treating sewage now dumped into the Ohio River. The courts held, however, that water users could not be made to pay for a service they were not getting. The appellate court said: "A municipality may not simply adopt an ordinance fixing rates and charges to be paid by the users of water under penalty of having their water supply discontinued on nonpayment, and the municipality taking no further steps in the completion of the proposed improvement for an indefinite time."

Water rights—South Dakota. A state water resources commission has been established to pass on all applications for water rights, inspect storage facilities, license well drillers, and keep records on all wells drilled. The authorizing statute declares: "All water within the state is the property of the people of the state. The state shall determine in what way the water of the state should be developed for the greatest public benefit." Previously water rights had rested on historical usage, with the landowner holding title to all water standing on his land or beneath it. Companies and individuals with vested interests existing prior to the effective date of the statute (Jul. 1, 1955) are protected by a clause in the law.—*Water Conditioning Reporter, May 26, 1955.*

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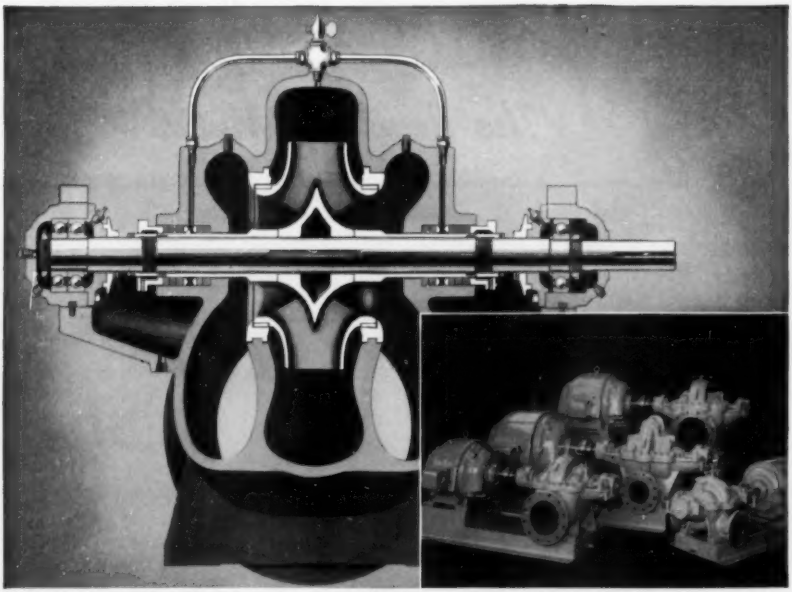

Cross Section Type M Pump. This construction is used, with minor variations, in all sizes 4" and larger.

Group of Double Suction Pumps ready for installation in municipal water works.

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This cut-away section of a Wheeler-Economy Type "M" Pump shows engineering that means around-the-clock, around-the-calendar performance . . . peace of mind for the men in charge. Longer life and less maintenance work results from these important features . . . The method of supporting the rotor between outboard bearings with end-thrust practically eliminated . . . horizontally split casing with all pipe connections in lower half so that rotating parts can be removed without disturbing connections or pump alignment . . . Inlet designed to reduce leakage and friction losses . . . Double-bearing construction provides for longer packing life . . . Low pressure stuffing boxes . . . Wearing parts readily renewable.

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The Reading Meter

Water for the Cities: *A History of the Urban Water Supply Problem in the United States. Nelson M. Blake. Syracuse University Press, 920 Irving Ave., Syracuse 10, N.Y. (1956) 341 pp.; $4*

This book by a university history professor relates the difficulties attending the establishment of public water systems in the cities of Philadelphia, New York, Boston, and Baltimore during the period 1790–1860. The systems were established largely to control fire and yellow fever, with individual convenience of secondary importance. The struggles were heightened by indecision over private and public ownership. Because of the personal ambition of a small group of influential men to establish a bank under the guise of providing a water system for New York City, the securing of an adequate and safe supply there was retarded for many years.

The work is thoroughly documented and is written in an interesting style

(*Continued on page 76 P&R*)

Be prepared for emergencies!

MANUAL FOR WATER PLANT OPERATORS

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General Considerations • Ground Water • Wells • Surface Water • General Aims and Methods of Water Treatment • Storage • Aeration • Coagulation and Sedimentation • Filtration • Disinfection • Iron Removal • Softening • Corrosion Control • Taste and Odor Removal • Pumping • Reservoirs • Piping and Connections • Hydraulic Measurement Devices • Plant Arithmetic • Chemical Tests • Water Bacteriology • Operation During Emergencies • Records and Operator Policy • Useful Literature • Useful Water Plant Data and Factors.

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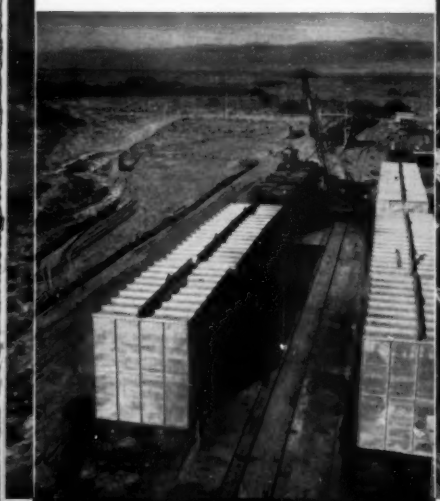
In making reinforced concrete pipe for water supply lines~



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Concrete pipe for main water supply lines, storm and sanitary sewers, subaqueous pipe lines

The Reading Meter

(Continued from page 74 P&R)

which almost permits the leading characters to live again. The activities of such outstanding engineers as Latrobe, Davis, Graff, Douglass, Jervis, Eddy, Chesborough, Whitwell, Sickels, Slade, and Manning are adequately presented. Truly, here is the story of water supply during an early and important period in this country's history. The many problems facing these early engineers convinced them of the need to go beyond the corporate limits for water and of the importance of an unpolluted source of water for urban living.

This book should be required reading for every student of public water supply.

Report of the Investigation of Leaching of a Sanitary Landfill. *Publication No. 10, State Water Pollution Control Board, Room 610, 721 Capitol Ave., Sacramento, Calif. (1954) 92 pp.; paperbound; free*

The effect on ground water of sanitary-landfill waste decomposition and leaching of soluble salts was the subject of an intensive 2-year field investigation conducted at Riverside, Calif. The results, which are given in detail, indicate that a landfill in intermittent or continuous contact with ground water will cause the water in the immediate vicinity to become grossly polluted. Dissolved mineral matter entering the ground water will undergo limited vertical diffusion (if the aquifer is 100 ft thick or more, the bottom water will probably remain unimpaired). When the pollutional load on

(Continued on page 78 P&R)

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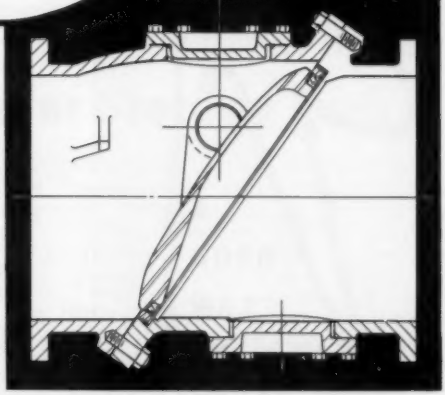

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When the flow is on, specially designed "airfoil" disc balances perfectly in open position. Fluid holds it tightly against stops. There's no vibration . . . no flutter. It's quiet. And note the design. Ample room is allowed around disc to assure low flow resistance.

When the flow is reversed, disc drops surely, quietly, *tightly* on special beveled seat. There's no banging, no slamming, no scraping or wearing of disc and seat faces, no damage to piping joints or the valve itself. It's as simple as that. It's as *sound* as that. It has the right idea.

Complete information on these rugged, quiet, long life check valves is highlighted in our Catalog 30-A. Whether you want iron or steel construction . . . whether you handle fluids or gases under a wide range of pressures . . . this catalog gives you all the facts. Send for your up-to-date copy, today.

The CHAPMAN VALVE Mfg. CO.

INDIAN ORCHARD, MASS.

The Reading Meter

(Continued from page 76 P&R)

a ground water is light by reason of intermittent and partial landfill contact, the most serious impairment of the ground water as little as half a mile from the landfill will be an increase in hardness, and then only in the upper portions of the aquifer.

A
GOOD
LAW
TO
BREAK
The law of averages says that cancer will kill one out of every six Americans. But the law does not have to prevail. You can help break it in two ways. By having a thorough medical checkup every year ... and by sending a contribution to your Unit of the American Cancer Society or to Cancer, c/o your town's Postmaster.
Fight cancer with a checkup and a check.
AMERICAN CANCER SOCIETY

Report on the Investigation of Travel of Pollution. *Publication No. 11, State Water Pollution Control Board, Room 610, 721 Capitol Ave., Sacramento, Calif. (1954) 218 pp.; paperbound; free*

In this 4-year study by the University of California, directed by Prof. Harold B. Gotaas, with P. H. McGauhey and Ray Krone conducting the experimental work, sewage effluent of various strengths was injected into a recharge well. The principal conclusions were that: bacteria travel rapidly at the start of recharge, reaching some maximum distance at rates governed by the induced ground water velocity; decrease of bacteria with distance of travel is rapid, the rate of removal depending on aquifer characteristics and not rate of recharge; suspended solids and organic matter form a mat that clogs the aquifer face and filters out bacteria; such clogging, however, increases the pressure necessary to inject sewage, but this difficulty can be overcome by injecting chlorine and then pumping out the well at twice the sewage injection rate, which was about 8.4 gpm per foot of aquifer.

A Glossary of Terms in Nuclear Science and Technology. *American Society of Mechanical Engineers, New York (1955) 189 pp.; $5*

Prepared under the direction of the National Research Council, this glossary incorporates in one alphabetical list the nuclear-science terms applicable to various branches of learning and technology. The publishers hope that this document will eventually be accepted as a standard throughout the world. Meanwhile it can serve as a helpful guide through the terminological maze of a field that is steadily growing in importance.

WEST WARWICK, R. I. — The Kent County Water Authority is owner of this 1,000,000-gal welded-steel reservoir which was put into service in 1955. The tank measures 52 ft in height by 58 ft in diameter. General contractor for the job was General Engineering & Contracting Co.; engineer was Charles A. Maguire & Associates.

AKRON, O.—A Bethlehem erection crew completed this attractive, 1,500,000-gal standpipe during the summer of 1955. Standing 115 ft high by 47 ft in diameter, it has an elliptical self-supporting roof and is dressed up with an ornamental cornice. Engineering was by the Bureau of Water Supply, W. R. LaDue, Chief Engineer and Superintendent.

Three New Water Storage Tanks

Welded steel tanks have many advantages for the storage of water above ground. They can be made permanently watertight; they are unaffected by climatic changes; they withstand damage due to shock, impact, explosion or surface settlement; they require little maintenance, and often cost less than competitive types of storage.

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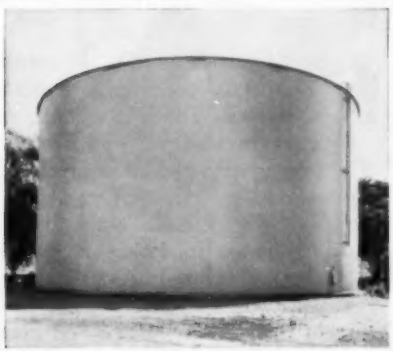

ROCHESTER, N. Y.—This 1,500,000-gal welded steel reservoir was built for New York Water Service Corp., which engineered the job. It is 40 ft high by 80 ft in diameter.

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The strongest pipe for water and sewage transmission. Leakproof. 40-ft lengths. Diameters 18 in. and up. For decades the preference of leading cities. May we give you more information?

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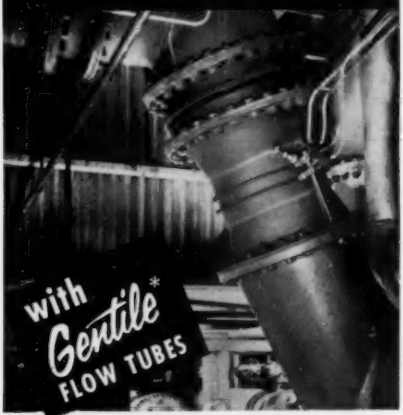
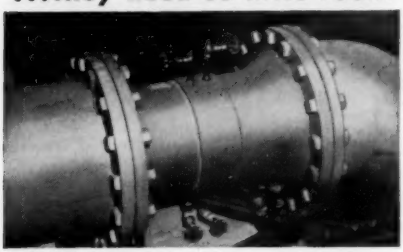


...they need so little room



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Gentile Flow Tubes are short. They need only minimum straight runs entering and following, and can be installed at practically any accessible point where flow conditions are reasonably steady.

Flow Tubes are furnished with individual head capacity curves - and for unusual piping arrangements, calibration curves for simulated conditions can be furnished.

Guaranteed Accuracy • Reproducibility
Minimum Head Loss

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AUTOMATIC VALVES • CONTROL VALVES
SAFETY VALVES • FLOW TUBES

(*Continued from page 54 P&R*)

Plastic pipe identification as safe for use in systems transporting drinking water is being carried on by the National Sanitation Foundation, a nonprofit research organization with headquarters at the School of Public Health, University of Michigan, Ann Arbor. Plastics manufacturers desiring the foundation's seal of approval must agree to meet minimum established standards and to permit regular inspection of their plants, as well as surprise sampling, by foundation officials. Uniform marking of all pipe and fittings will assure users of conformance.

The identification program has been developed with the cooperation of producers of plastic raw materials and pipe through the Society of the Plastics Industry, New York. The SPI underwrote the cost of a National Sanitation Foundation long-term study on toxicity and taste in water supplied through plastic pipe (see August 1954 JOURNAL, pp. 775–787). The SPI also has research under way, at another laboratory, on physical properties, performance characteristics, and testing of plastic pipe.

Neptune Meter Co. is continuing its expansion program by combining with Eastern Industries, Inc., manufacturer of radar speed meters and vehicle-actuated traffic controls. Eastern Industries, which has plants in Hamden and East Norwalk, Conn., was founded in 1946 by Eugene D. Stirlen, president and chairman of the board. The company's other products include small-size pumps and hydraulic equipment, servo systems, and newly developed instruments to pressurize and cool electronic equipment. Eastern also makes mixing equipment and plastic products.

(*Continued on page 82 P&R*)

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(Continued from page 80 P&R)

Ralph B. Cooney, vice-president of Kelly, Nason, Inc., advertising agency, New York, died Feb. 21, 1956, at the age of 57. A graduate of Indiana University, he began his advertising career with L. S. Ayres & Co., an Indianapolis department store, later becoming public relations director of Gimbel Brothers in Milwaukee and of E. T. Slattery Co. in Boston. Before joining Kelly, Nason in 1938, he was director of advertising for the real estate firm of Bing & Bing, New York.

Mr. Cooney had long maintained an interest in utility public relations. He contributed a number of papers to AWWA meetings and was the author of the series, "Silent Service Is Not Enough!" which has since been incorporated in an AWWA manual.

John J. Harte, president, John J. Harte Co., architectural engineers, Atlanta, died Nov. 6, 1955, after a 2-month illness. He was 52. A former associate of Robert & Co. and McCrary Engineering Corp., both of Atlanta, and a consulting engineer to the government of Ecuador, Mr. Harte organized the firm which bears his name in 1941. He was connected with the design and construction of many large installations in the Southeast, including the chemical plant for the US Army which is located at Kingsport, Tenn.

An AWWA member since 1945, Mr. Harte also belonged to NSPE, the American Institute of Industrial Engineers, and the American Institute of Management.

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CLAYTON 5, N. J.

THE H & T
POPPET TYPE
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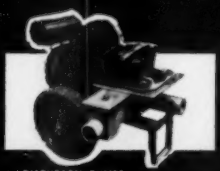

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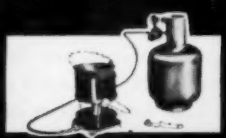

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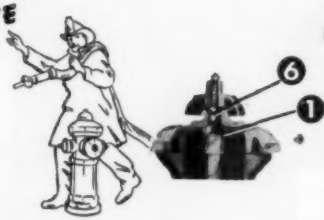
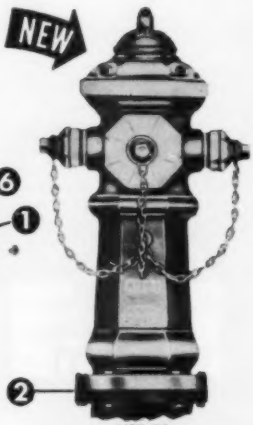


NEW



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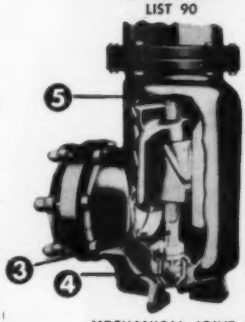

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CHECK VALVES • TAPPING SLEEVES • AIR RELEASE VALVES

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16

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VALVE CO. **TROY, NEW YORK**

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GATE VALVES • FIRE HYDRANTS • SQUARE BOTTOM VALVES
CHECK VALVES • TAPPING SLEEVES • AIR RELEASE VALVES

NEW MEMBERS

Applications received Feb. 1–29, 1956

Albert, Paul McHenry, Cons. Engr., 206 S. Main St., Ann Arbor, Mich. (Jan. '56) *PD*

Alger, Charles G., Project Engr., H. E. Bovay, Jr., Cons. Engrs., 718 Spokane & Eastern Bldg., Spokane 1, Wash. (Jan. '56) *D*

Alves, Harry R., City Treas., Box 644, New Braunfels, Tex. (Jan. '56) *M*

Alway, Donald L., Chief Civ. Engr., Victor Green & Assocs., Inc., 2100 Industrial Bldg., Detroit 26, Mich. (Jan. '56) *PD*

Anderson, Robert R., Pres., Settle Engrs. Inc., Box 2821, Charleston, W.Va. (Jan. '56) *MRPD*

Anderson, S. Pickens, Engr., Piedmont Eng. Service, 508 E. North St., Greenville, S.C. (Jan. '56) *R*

Arraj, Robert E.; *see* Kimball (Neb.)

Ashby, Glenn, Constr. Supt., Water Works Dept., 400 N.W. Riverside Dr., Evansville, Ind. (Jan. '56) *R*

Athenes, Piree & Environs, Societe Anonyme Hellenigue des Eaux D', A Georgolas, Mgr.-Director, Kolokotroni 4, Athens, Greece (Corp. M. Jan. '56) *MRPD*

Aycock, Milton H., Dist. Mgr., Centriline Corp., 1526 Healey Bldg., Atlanta 3, Ga. (Jan. '56) *MD*

Banker, Robert F., Civ. Engr., Black & Veatch, 4706 Broadway, Kansas City 12, Mo. (Jan. '56) *MRP*

Barnett, Reo V., Secy. & Purchasing Agent, West Virginia Water Service Co., Box 1906, Charleston, W.Va. (Jan. '56) *MR*

Barr, George W., Engr., Yost & Gardner Engrs., 194 N. Meyer, Tucson, Ariz. (Jan. '56) *MRPD*

Becher, Archie E., Pres., Becher-Hoppe Engrs. Inc., 1109 Prospect Ave., Wausau, Wis. (Jan. '56) *MRPD*

Bell, Howard Otis, Water Supt., Yellville, Ark. (Jan. '56) *MRPD*

Bigham, Roger Vance, Sales Engr., Steam Specialties Co., 15794 Wyoming Ave., Detroit 38, Mich. (Jan. '56)

Bird, C. St. John, Cons. Engr., 22 Ameshoff St., Braamfontein, Johannesburg, South Africa (Jan. '56) *RPD*

Boulahanis, A. J., Engr., South Cook County Mosquito Abatement Dist., Box 338, Posen, Ill. (Jan. '56)

Boulton, Edward J., Mgr., Citizens Water Service Co., Philipsburg, Pa. (Jan. '56)

Bowman, Gilbert T., Mgr., Products Dept., Rockwell Mfg. Co., 400 N. Lexington Ave., Pittsburgh 8, Pa. (Jan. '56) *D*

Bowser, Reginald B., Public Health Engr., Maint. Div., Directorate of Works, Army Headquarters, Ottawa, Ont. (Jan. '56)

Boyer, Douglas L., Special Repr., International Salt Co., Inc., 2016 Haynes Ave., Indianapolis, Ind. (Jan. '56) *P*

Bragg, O. E., Turbine Mechanic, Water Works, 651—14th St., N.W., Atlanta 13, Ga. (Jan. '56) *M*

Bridgford, Bernard Reed, Sewer & Water Engr., Johnson & Anderson Inc., 1307 Pontiac State Bank Bldg., Pontiac, Mich. (Jan. '56) *RD*

Brown, Leonard L., Supt., Water & Sewer Dept., 116 S. Spring St., Harrison, Ark. (Jan. '56) *MP*

Brown, Robert L., Cons. Engr., Black & Veatch, 4706 Broadway, Kansas City 12, Mo. (Jan. '56)

Brown, Roy L., Asst. San. Engr., State Health Dept., Bureau of San. Eng., Richmond 19, Va. (Jan. '56) *P*

Bukraba, Piotr, Engr., Greeley & Hansen, 220 S. State St., Chicago 4, Ill. (Jan. '56)

Bush, Wesley A., Cons. Engr., 7100 W. Ocean Front, Newport Beach, Calif. (Jan. '56)

Cape May, City of, Carl R. Youngberg, Comr., Public Works, Cape May, N.J. (Corp. M. Jan. '56)

Carson, Thomas O., Dist. Mgr., Rockwell Mfg. Co., 390 N. Lexington Ave., Pittsburgh 8, Pa. (Jan. '56) *PD*

Catt, Paul L., Office Mgr., Ralph D. Green Constr. Co., Inc., 817 Barnett, Vincennes, Ind. (Jan. '56) *D*

Chiang, Chai-Sung, Graduate Student, Massachusetts Inst. of Technology, Rm. 1—153, Cambridge 39, Mass. (Jr. M. Jan. '56) *MPD*

Cooney, Robert J., Sr. Engr., Indianapolis Water Co., 113 Monument Circle, Indianapolis, Ind. (Jan. '56) *MRPD*

Costello, Charles V., Chief Filter Operator, Bureau of Water, 305 Court St., Binghamton, N.Y. (Jan. '56) *P*

Cross, Arthur D., Jr., Asst. Mgr., Mount Holly Water Co., 84 Mill St., Mount Holly, N.J. (Jan. '56) *M*

Curtis, Elbert D., Eng. Supervisor, Consolidated Western Steel Div., Box 2015 Terminal Annex, Los Angeles 54, Calif. (Jan. '56)

Dabel, William J., Asst. City Engr., Box 660, Napa, Calif. (Jan. '56) *MP*

Deansboro Water Co., George H. Skinner, Pres., Deansboro, N.Y. (Corp. M. Jan. '56) *MR*

Dearborn, Township of, George E. Tennant, Mgr., Water Dept., 26807 Michigan Ave., Inkster, Mich. (Corp. M. Jan. '56) *MD*

Dellman, Milo, Comr., Public Works, South Gate, Calif. (Jan. '56) *MR*

Diamond, Lawrence, Comr., Passaic Valley Water Com., 1525 Main Ave., Clifton, N.J. (Jan. '56) *M*

Donaldson, Francis H., Asst. Div. Mgr., American Water Works Service Co., Inc., 14½ N. 10th St., Richmond, Ind. (Jan. '56) *M*

Dunne, Glenn M., Jr., Diretcor, Hutchinson Water Co., 509 N. Main, Hutchinson, Kan. (Jan. '56) *M*

Ely, Paul M., Supt., Water Dept., Arcata, Calif. (Jan. '56) *PD*

Erwin, R. Blake, Cons. Engr., Ontario Land Surveyor, 437 Queen St., Niagara Falls, Ont. (Jan. '56)

Etzel, James E., Instructor, San. Eng., Purdue Univ., West Lafayette, Ind. (Jan. '56) *PD*

Evenson, Kenneth E., Foreman, Water Dept., City Hall, Kent, Wash. (Jan. '56) *D*

Fenerdjian, Hrant A., Chief Engr., Societe Anonyme Hellenigue des Eaux, D'Athenes Piree & Environs, Kolokotroni 4, Athens, Greece (Jan. '56)

Georgolas, A.; *see* Athenes, Piree & Environs, Societe Anonyme Hellenigue des Eaux D'

Gildard, J. H., Sales Engr., B-I-F Pacific, Inc., 1036 University Ave., Berkeley 2, Calif. (Jan. '56) *P*

Greathouse, Edward D., Civ. Engr., Hill & Ingman, 1728 E. Madison St., Seattle, Wash. (Jan. '56) *RD*

Gunder, Delbert, City Engr., Water Works Dept., Evansville, Ind. (Jan. '56) *MRP*

Gutierrez, Raul, Jr., Mfer.'s Repr., Anderson Trading Co., Obispo 355, Havana, Cuba (Jan. '56)

Haag, Gerald, Public Works Director, Greendale, Wis. (Jan. '56) *MR*

Hammarlund Mfg. Co., Inc., Frederick F. Sylvester, Gen Sales Mgr., 460 W. 34th St., New York 1, N.Y. (Assoc. M. Jan. '56)

Hancock, John C., Civ. Engr., National Sanitation Foundation, Ann Arbor, Mich. (Jan. '56) *D*

Hartke, Loma W.; *see* Huntingburg (Ind.) Munic. Light & Water Plant

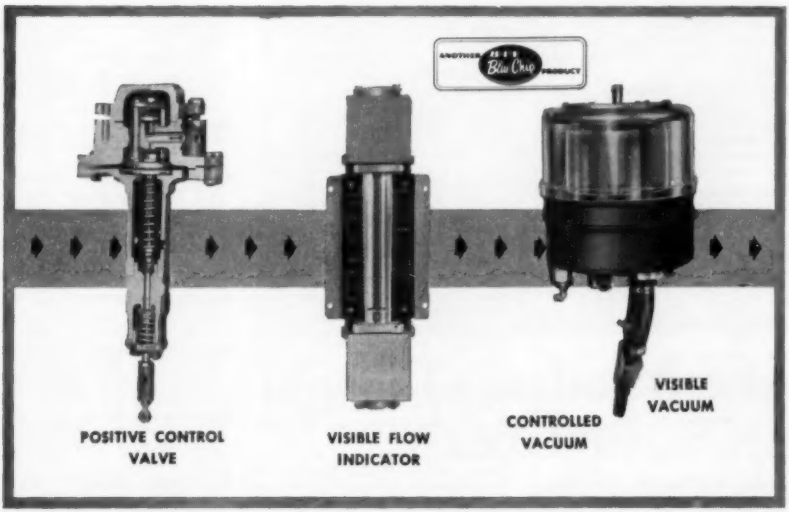
Hayden, Harry N., Supt., Water System, 379 El Camino Real, Millbrae, Calif. (Jan. '56) *M*

Hinchliffe, John V., Comr., Passaic Valley Water Com., 1525 Main Ave., Clifton, N.J. (Jan. '56) *M*

Hodgkinson, Rulon, Asst. Supt., California Water & Telephone Co., 481 Tyler St., Monterey, Calif. (Jan. '56) *D*

Howe, Raymond L., Borough Mgr., Municipal Bldg., Sewickley, Pa. (Jan. '56) *MD*

(Continued on page 88 P&R)



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(Continued from page 86 P&R)

Huntingburg Munic. Light & Water Plant, Loma W. Hartke, Supt., 6th & Van Buren, Huntingburg, Ind. (Corp. M. Jan. '56) *RPD*

Hurst, Robert Eugene, Supt., Water System, Mountain Home, Ark. (Jan. '56) *M*

Irwin, Robert M., Asst. Mgr., Water Works Dept., 36 W. 2nd St., Chillicothe, Ohio (Jan. '56) *MRPD*

Jarboe, Richard J., Exec. Secy., Water Works Dept., City Hall, 3rd & Walnut Sts., Evansville, Ind. (Jan. '56) *M*

Jeris, John S., Development Engr., US Army Eng. Research & Development Lab., Fort Belvoir, Va. (Jan. '56) *RP*

Johnston, Clark B.; *see* Mather Water Co.

Judge, James H., Asst. to the Pres., American Meter Co., Inc., Philmont & Bustleton Aves., Philadelphia 16, Pa. (Jan. '56)

Karlson, Wesley N., Water Engr., Alexander Chem. Corp., 35 E. Wacker Dr., Chicago 1, Ill. (Jan. '56) *P*

Kennedy, William S., Vice-Pres., Snyder Co., Inc., 1432 E. 6th St., Tulsa 20, Okla. (Jan. '56) *M*

Keylon, D. A., Office Engr., Johnson & Anderson Inc., 1307 Pontiac State Bank Bldg., Pontiac, Mich. (Jan. '56) *MRPD*

Killian, Lawrence, Filter Supt., Water Works Dept., Shawnee Dr., Evansville, Ind. (Jan. '56) *P*

Kilpatrick, Henry, Water Comr., St. Paul, Minn. (Jan. '56) *MD*

Kimball, City of, Robert E. Arraj, City Engr., City Office, Kimball, Neb. (Munic. Sv. Sub. Jan. '56) *MRD*

Kroll, Max, Comr., Passaic Valley Water Com., 1525 Main Ave., Clifton, N.J. (Jan. '56) *M*

Kubota, Jack Yasuo, Asst. San. Engr., State Dept. of Public Health, Bureau of San. Eng., Rm. 7, B St. Pier, San Diego 1, Calif. (Jan. '56) *P*

Larson, Richard W., Sales Engr., Johns-Manville Sales Corp., Box 595, Saginaw, Mich. (Jan. '56) *D*

Lewis, Arthur H., Assoc., Consoer, Townsend & Assocs., 360 E. Grand Ave., Chicago 11, Ill. (Jan. '56) *PD*

Liljegren, Goran G. J., Assoc., Consoer, Townsend & Assocs., 351 E. Ohio St., Chicago 11, Ill. (Jan. '56) *RP*

Lindstrom, Carl A., Jr., San. Engr., Indian Health Service, US Public Health Service, 1011 N. 2nd St., Aberdeen, S.D. (Jan. '56) *PD*

Lusk, Donald G., Sr. Sanitarian, Water & Sewerage, Los Angeles County Health Dept., 241 N. Figueroa, Los Angeles, Calif. (Jan. '56) *RPD*

Lyons, Gerald S., Asst. Engr., Indianapolis Water Co., 113 Monument Circle, Indianapolis, Ind. (Jan. '56) *MRPD*

Manning, K. S., Mgr. Venezuela Gulf Refining Co., Apt. 16, Barcelona, Venezuela (Jan. '56) *MPD*

Mansfield, Gordon, Borough Engr., Hightstown Borough, N.J. (Jan. '56) *MD*

Martin, Everett V., Director, Nankin Township Water Dept., 32715 Dorsey Rd., Wayne, Mich. (Jan. '56)

Mason, Richard F., Utility Engr., Maine Public Utilities Com., State House, Augusta, Me. (Jan. '56) *P*

Mather Water Co., Clark B. Johnston, Pres., Box 548, Fredericktown, Pa. (Corp. M. Jan. '56)

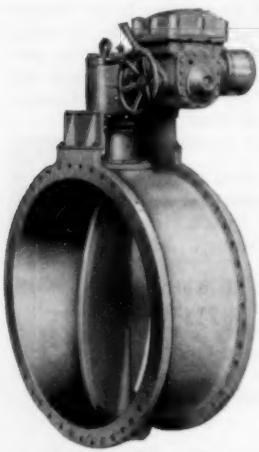
McDonald, James E., Civ. Engr., James E. McDonald Co., 1315 Pembroke Ave., N.E., Knoxville 17, Tenn. (Jan. '56) *MP*

Mileo, Anthony J., Engr., National Board of Fire Underwriters, 85 John St., New York 38, N.Y. (Jan. '56) *PD*

Moorhouse, Ralph F., Chief Constr. Engr., Consoer, Townsend & Assocs., 351 E. Ohio St., Chicago, Ill. (Jan. '56) *PD*

Mortenson, LeRoy A., Sales Engr., Turbine Equipment Co., 63 Vesey St., New York, N.Y. (Jan. '56)

(Continued on page 90 P&R)



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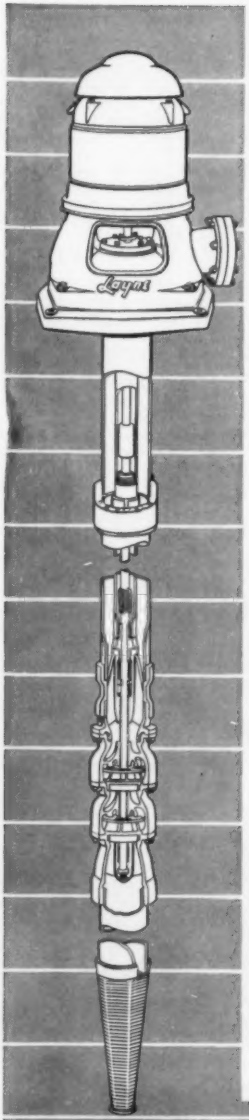

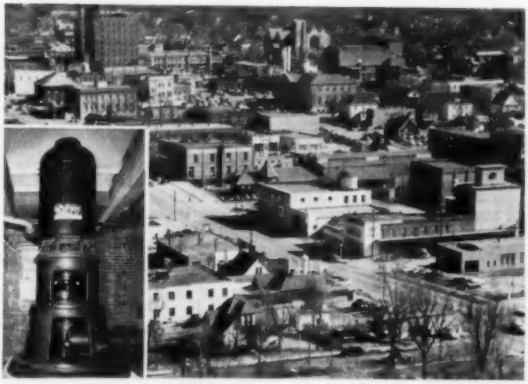

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(Continued from page 88 P&R)

Moses, Thomas W., Asst. to the Pres., Indianapolis Water Co., 113 Monument Circle, Indianapolis 6, Ind. (Jan. '56) *M*

Nevers, Rolfe K., Accountant & Purchasing Agent, Water & Light Dept., Moncton, N.B. (Jan. '56) *M*

Newell, I. Laird, Cons. Engr., Henry Souther Eng. Co., 11 Laurel St., Hartford, Conn. (Jan. '56) *RPD*

Orchard, Melvin E., Civ. Engr., Johnson & Anderson Inc., 1307 Pontiac State Bank Bldg., Pontiac, Mich. (Jan. '56) *D*

Ordon, Chester J., Assoc. Prof., Dept. of Civ. Eng., Wayne Univ., Detroit 1, Mich. (Jan. '56) *RPD*

Osawatomie, City of, L. E. Teeter, Mayor, 1035 Main St., Osawatomie, Kan. (Corp. M. Jan. '56)

Palmer, Frank, Supt., West Slope Water Dist., 3105 S.W. 89th Ave., Portland 1, Ore. (Jan. '56) *D*

Pauls, Harry G., Engr., Greeley & Hansen, 220 S. State St., Chicago 4, Ill. (Jan. '56)

Petitfils, Bryan T., Secy., Water Works Dist. No. 2, St. Bernard, La. (Jan. '56) *M*

Pickens, M. Lee, Supt., California-Michigan Land & Water Co. & East Pasadena Water Co., Ltd., 269 S. Rosemead Blvd., East Pasadena 10, Calif. (Jan. '56) *MRPD*

Purcell, W. C., Township Engr., Atikokan, Ont. (Jan. '56) *MD*

Putnam, William H. H., Gen. Mgr., Water Works Board, Birmingham, Ala. (Jan. '56) *M*

Safford, Francis I., Financial Analyst, R. L. Day & Co., 75 Pearl St., Hartford 3, Conn. (Jan. '56) *PD*

Sayles, Dwight R., Engr., Black & Veatch, 4706 Broadway, Kansas City 12, Mo. (Jan. '56) *P*

Severin, Clifford E., Salesman, Johns-Manville Sales Corp., Box 871, Spokane 10, Wash. (Jan. '56) *D*

Shell, Billy J., Civ. & San. Engr., San Xavier R. & S. Co., Box 1031, Tucson, Ariz. (Jan. '56) *PD*

Shull, Lardner H., Pres., Ansonia Water Co., Ansonia, Conn. (Jan. '56) *M*

Skinner, George H.; *see* Deansboro (N.Y.) Water Co.

Smith, Donald B., Supt., Board of Public Affairs, Box 2575, Whitehouse, Ohio (Jan. '56) *M*

Smith, Elbert T., Plant Engr., Pure Water Co., 728 Stratton St., Logan, W.Va. (Jan. '56) *MP*

Smith, Frank G., Mgr., Sales Dept., Empire Brass Mfg. Co. Ltd., Dunda St. E., London, Ont. (Jan. '56)

Snyder, George E., Cons. Civ. Engr., 511 S. Jackson St., Jackson, Mich. (Jan. '56) *RP*

Sochowicz, George A., Sr. Asst. Civ. Engr., Dept. of Water Supply, 735 Randolph St., Detroit, Mich. (Jan. '56) *PD*

Sprague, S. B., Supt., Board of Public Affairs, Hudson, Ohio (Jan. '56) *MRPD*

Sterba, Richard E., Sales Repr., Johns-Manville Sales Corp., Box 2012, South Bend, Ind. (Jan. '56) *D*

Steytler, Robert B., Engr., Bureau of Water, Box 1729, Perry Square Sta., Erie, Pa. (Jan. '56)

Strom, Richard M., Sales Engr., Transite Pipe Div., Johns-Manville Sales Corp., 915 Terminal Sales Bldg., Portland 5, Ore. (Jan. '56) gart N., Germany (Corp. M. Jan. '56) *MRPD*

Swanson, John J., Utilities Supervisor, US Naval Air Sta., Olathe, Kan. (Jan. '56)

Swets, Walter, Supt., Water Dept., 3534 Ridge Rd., Highland, Ind. (Jan. '56) *MD*

Sylvester, Frederick F.; *see* Hammarlund Mfg. Co.

Tait, W. S., Sr. Engr., P.H.E. Div., Dept. of National Health, 4th Floor, Dominion Bldg., Queen St., St. Catherines, Ont. (Jan. '56)

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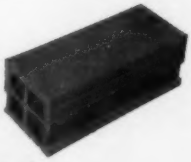

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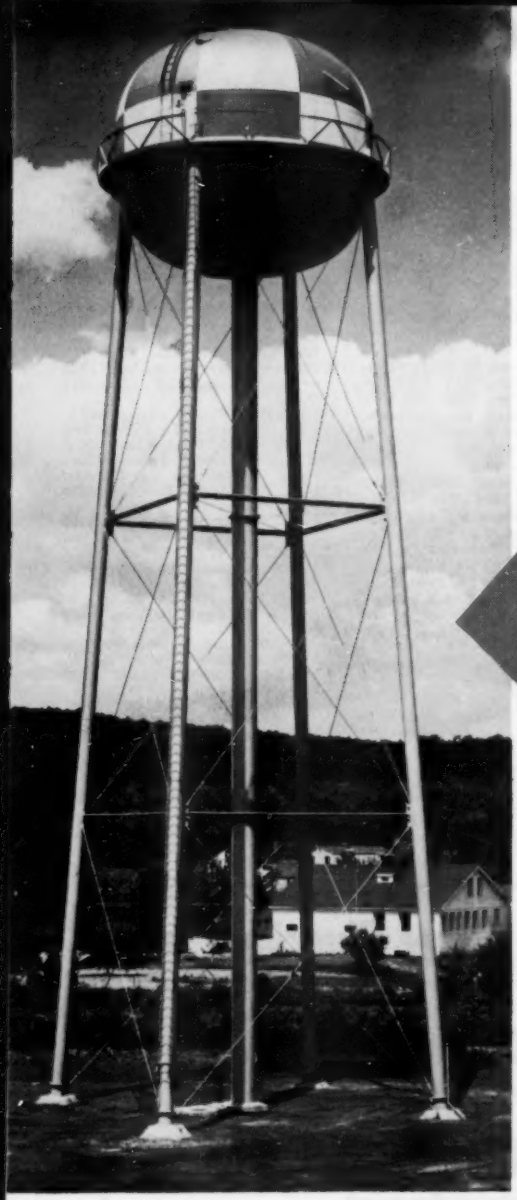

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Tamone, Benjamin J., Foreman, Water Dept., 500 Benton St., Santa Clara, Calif. (Jan. '56) *MD*

Teeter, L. E.; *see* Osawatomie (Kan.)

Tennant, George E.; *see* Dearborn (Mich.)

Thompson, I. H., Supt., Parkin Water Co., Parkin, Ark. (Jan '56) *P*

Thompson Kenneth G., Town Engr., Truro, N.S. (Jan. '56) *MRPD*

Thomsen, Charles D., Prin. Asst. Engr., Greeley & Hansen, 220 S. State St., Chicago 2, Ill. (Jan. '56) *PD*

Tobey, Frank T., Jr., Owner, Frank T. Tobey Jr. & Assocs., 707 Falls Bldg., Memphis, Tenn. (Jan. '56) *MRPD*

Torkelson, Carl, Supt., Water Dept., Box 123, Grafton, N.D. (Jan. '56) *MRP*

Trail, James Herbert, Jr., Student, Univ. of Illinois, 108 N. Romine St., Urbana, Ill. (Jr. M. Jan. '56)

Uray, Charles, Jr., Engr. & Surveyor, H. W. Scott & Assoc., 6090 S.O.M. Center Rd., Solon, Ohio (Jan. '56) *MRPD*

Vecchiarelli, Francis, Supt., Eng. & Research, Hunter Douglas Aluminum Corp., Flemington, N.J. (Jan. '56) *RP*

Vogh, Richard P., Supt., Water Plant, Gainesville, Fla. (Jan. '56) *MPD*

Wakeman, Norman B., Editorial Director, Gebbie Press, 19 E. 48th St., New York, N.Y. (Jan. '56) *RD*

Wallace, Sterlin, Supt., Munic. Water System, Marvell, Ark. (Jan. '56) *MD*

Walton, Gordon L., Water Supt., 500 Hamilton Rd., Glassboro, N.J. (Jan. '56) *RPD*

Warner, William S., Engr., Bridgeport Hydr. Co., 835 Main St., Bridgeport, Conn. (Jan. '56) *MRD*

Warren, Richard W., Contracting Engr., Chicago Bridge & Iron Co., 332 S. Michigan Ave., Chicago 4, Ill. (Jan. '56) *M*

Weaver, Fayette, Supt., Sharon Springs, Kan. (Jan. '56) *MRPD*

Weimer, William F., Adv. Mgr., Rockwell Mfg. Co., 400 N. Lexington Ave., Pittsburgh 8, Pa. (Jan. '56) *MRPD*

Weinberg, Lester H., Cons. Engr., 405 Portage Trail, Cuyahoga Falls, Ohio (Jan. '56) *RPD*

Weir, John M., Chief Engr., Dept. of Public Property, Water Dept., 404 County Bldg., Decatur, Ill. (Jan. '56) *MRP*

Wheten, W. A., Water Works Design Engr., Engrs. Dept., 44 Hughson St., N., Hamilton, Ont. (Jan. '56)

Wiedensohler, Andrew, Supt., Dept. of Public Works, 19617 Harper Ave., Harper Woods 36, Mich. (Jan. '56) *D*

Wilderman, C. L., Jr., Chief Engr., Kelly Well Co., 617 E. 7th, Grand Island, Neb. (Jan. '56) *RPD*

Williams, G. A., Supt. of Utilities, Drawer 400, The Pas, Man. (Jan. '56)

Williams, Raymond L., Business Development Repr., Indianapolis Water Co., 113 Monument Circle, Indianapolis, Ind. (Jan. '56) *M*

Williamson, Robert S., Supt., Water Dept., Dumas, Ark. (Jan. '56) *M*

Willis, Roderick M., Pres., Northwest Filter Co., 122 Elliott Ave. W., Seattle 99, Wash. (Jan. '56) *P*

Wilson, C. Glenn, Liaison Engr., Freese & Nichols, 407 Danciger Bldg., Fort Worth, Tex. (Jan. '56) *P*

Woods, Sidney W., Engr., 301–305 Lister Bldg., Hamilton, Ont. (Jan. '56)

Wooten, Edward J., Chief Eng., River Pumping Sta., Water Dept., Atlanta, Ga. (Jan. '56) *M*

Youngberg, Carl R.; *see* Cape May (N.J.)

Zeitler, Braman E., Asst. Supt. of Water & Public Works, 30 W. Main St., Webster, N.Y. (Jan. '56) *MPD*

Zimmerman, E. Newt, Adv. Mgr., Chicago Bridge & Iron Co., 332 S. Michigan Ave., Chicago 4, Ill. (Jan. '56) *M*

Zoellner, Guenter E. O., Project Engr., Marshall, Macklin & Monaghan, 154 Merton St., Toronto 7, Ont. (Jan. '56)

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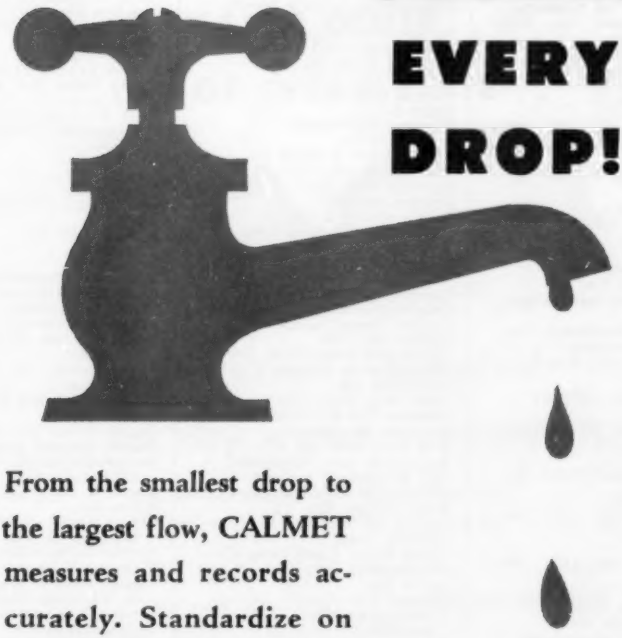

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We are proud to salute the American Water Works Association on their 75th year of progress and achievement in the Water Works field! And, we're proud for another reason—it's National's 50th Anniversary! For us, it means a half-century of progress increasing efficiency, carrying capacity and pressure of water mains *all over the world!*

Write us today for facts and information that shows how National's 50 years of experience cleaning water mains can restore your system to at least 95% of its original rated capacity.

You are cordially invited to visit our booths numbers 1 and 2 at the National Convention of the American Water Works Association, to be held in St. Louis, Missouri.

50TH

NATIONAL

ANNIVERSARY

WATER MAIN CLEANING CO.
50 Church St., New York 7, N. Y.

Permutit Co.
Roberts Filter Mfg. Co.
Walker Process Equipment, Inc.

Softening Chemicals and Compounds:
Calgon, Inc.
Cochrane Corp.
General Filter Co.
Infilco Inc.
Morton Salt Co.
Permutit Co.
Tennessee Corp.

Standpipes, Steel:
Chicago Bridge & Iron Co.
R. D. Cole Mfg. Co.
Graver Water Conditioning Co.
Hammond Iron Works
Pittsburgh-Des Moines Steel Co.

Steel Plate Construction:
Bethlehem Steel Co.
Chicago Bridge & Iron Co.
R. D. Cole Mfg. Co.
Graver Water Conditioning Co.
Hammond Iron Works
Pittsburgh-Des Moines Steel Co.

Stops, Curb and Corporation:
Hays Mfg. Co.
Mueller Co.

Storage Tanks; see Tanks

Strainers, Suction:
James B. Clow & Sons
M. Greenberg's Sons
Johnson, Edward E., Inc.
R. D. Wood Co.

Surface Wash Equipment:
Cochrane Corp.
Permutit Co.

Swimming Pool Sterilization:
Builders-Providence, Inc. (Div., B-I-F Industries)
Fischer & Porter Co.
Omega Machine Co. (Div., B-I-F Industries)
Proportioneers, Inc. (Div., B-I-F Industries)
Wallace & Tiernan Inc.

Tanks, Steel:
Bethlehem Steel Co.
Chicago Bridge & Iron Co.
R. D. Cole Mfg. Co.
Graver Water Conditioning Co.
Hammond Iron Works
Pittsburgh-Des Moines Steel Co.

Tapping-Drilling Machines:
Hays Mfg. Co.
Mueller Co.
A. P. Smith Mfg. Co.

Tapping Machines, Corp.:
Hays Mfg. Co.
Mueller Co.

Taste and Odor Removal:
Builders-Providence, Inc. (Div., B-I-F Industries)
Cochrane Corp.
Fischer & Porter Co.
General Filter Co.
Graver Water Conditioning Co.
Industrial Chemical Sales Div.
Infilco Inc.
Permutit Co.
Proportioneers, Inc. (Div., B-I-F Industries)
Wallace & Tiernan Inc.

Tenoning Tools:
Spring Load Mfg. Corp.

Turbidimetric Apparatus (For Turbidity and Sulfate Determinations):
Wallace & Tiernan Inc.

Turbines, Steam:
Allis-Chalmers Mfg. Co.
DeLaval Steam Turbine Co.

Turbines, Water:
Allis-Chalmers Mfg. Co.
DeLaval Steam Turbine Co.

Valve Boxes:
James B. Clow & Sons
Ford Meter Box Co.
M & H Valve & Fittings Co.
Mueller Co.
Rensselaer Valve Co.
A. P. Smith Mfg. Co.
Trinity Valley Iron & Steel Co.
R. D. Wood Co.

Valve-Inserting Machines:
Mueller Co.
A. P. Smith Mfg. Co.

Valves, Altitude:
Golden-Anderson Valve Specialty Co.
Ross Valve Mfg. Co., Inc.
S. Morgan Smith Co.

Valves, Butterfly, Check, Flap, Foot, Hose, Mud and Plug:
Builders-Providence, Inc. (Div., B-I-F Industries)
Chapman Valve Mfg. Co.
James B. Clow & Sons
DeZurik Shower Co.
M. Greenberg's Sons
Kennedy Valve Mfg. Co.
M & H Valve & Fittings Co.
Mueller Co.
Henry Pratt Co.
Rensselaer Valve Co.
S. Morgan Smith Co.
R. D. Wood Co.

Valves, Detector Check:
Hersey Mfg. Co.

Valves, Electrically Operated:
Builders-Providence, Inc. (Div., B-I-F Industries)
Chapman Valve Mfg. Co.
James B. Clow & Sons
Crane Co.
Darling Valve & Mfg. Co.
Golden-Anderson Valve Specialty Co.
Kennedy Valve Mfg. Co.
M & H Valve & Fittings Co.
Mueller Co.
Philadelphia Gear Works, Inc.
Henry Pratt Co.
Rensselaer Valve Co.
A. P. Smith Mfg. Co.
S. Morgan Smith Co.

Valves, Float:
James B. Clow & Sons
Golden-Anderson Valve Specialty Co.
Henry Pratt Co.
Ross Valve Mfg. Co., Inc.

Valves, Gate:
Chapman Valve Mfg. Co.
James B. Clow & Sons
Crane Co.
Darling Valve & Mfg. Co.
Dresser Mfg. Div.
Kennedy Valve Mfg. Co.
Ludlow Valve Mfg. Co., Inc.
M & H Valve & Fittings Co.
Mueller Co.
Rensselaer Valve Co.
A. P. Smith Mfg. Co.
R. D. Wood Co.

Valves, Hydraulically Operated:
Builders-Providence, Inc. (Div., B-I-F Industries)
Chapman Valve Mfg. Co.
James B. Clow & Sons
Crane Co.
Darling Valve & Mfg. Co.
DeZurik Shower Co.
Golden-Anderson Valve Specialty Co.
Kennedy Valve Mfg. Co.
F. B. Leopold Co.
M & H Valve & Fittings Co.
Mueller Co.
Philadelphia Gear Works, Inc.
Henry Pratt Co.
Rensselaer Valve Co.
A. P. Smith Mfg. Co.
S. Morgan Smith Co.
R. D. Wood Co.

Valves, Large Diameter:
Chapman Valve Mfg. Co.
James B. Clow & Sons
Crane Co.
Darling Valve & Mfg. Co.
Golden-Anderson Valve Specialty Co.
Kennedy Valve Mfg. Co.
Ludlow Valve Mfg. Co., Inc.
M & H Valve & Fittings Co.
Mueller Co.
Henry Pratt Co.
Rensselaer Valve Co.
A. P. Smith Mfg. Co.
S. Morgan Smith Co.
R. D. Wood Co.

Valves, Regulating:
DeZurik Shower Co.
Foster Eng. Co.
Golden-Anderson Valve Specialty Co.
Minneapolis-Honeywell Regulator Co.
Mueller Co.
Henry Pratt Co.
Ross Valve Mfg. Co.
S. Morgan Smith Co.

Valves, Swing Check:
Chapman Valve Mfg. Co.
James B. Clow & Sons
Crane Co.
Darling Valve & Mfg. Co.
Golden-Anderson Valve Specialty Co.
M. Greenberg's Sons
M & H Valve & Fittings Co.
Mueller Co.
Rensselaer Valve Co.
A. P. Smith Mfg. Co.
R. D. Wood Co.

Venturi Tubes:
Builders-Providence, Inc. (Div., B-I-F Industries)
Infilco Inc.
Simplex Valve & Meter Co.

Waterproofing:
Barrett Div.
Inertol Co., Inc.

Water Softening Plants; see Softeners

Water Supply Contractors:
Layne & Bowler, Inc.

Water Testing Apparatus:
Wallace & Tiernan Inc.

Water Treatment Plants:
American Well Works
Chain Belt Co.
Chicago Bridge & Iron Co.
Cochrane Corp.
Dorr-Oliver Inc.
Fischer & Porter Co.
General Filter Co.
Graver Water Conditioning Co.
Hammond Iron Works
Hungerford & Terry, Inc.
Infilco Inc.
Permutit Co.
Pittsburgh-Des Moines Steel Co.
Roberts Filter Mfg. Co.
Walker Process Equipment, Inc.
Wallace & Tiernan Inc.

Well Drilling Contractors:
Layne & Bowler, Inc.

Well Screens:
Johnson, Edward E., Inc.

Wrenches, Ratchet:
Dresser Mfg. Div.

Zeolite; see Ion Exchange Materials

A complete Buyers' Guide to all water works products and services offered by AWWA Associate Members appears in the 1955 AWWA Directory.

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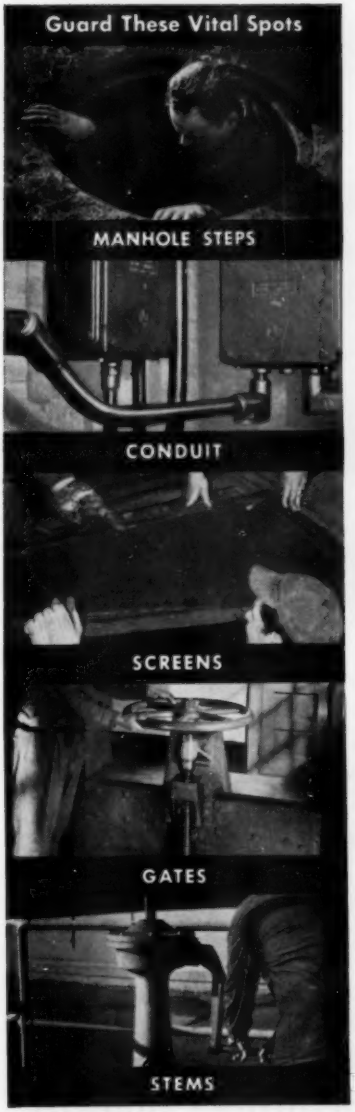

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EVERDUR IS TOUGH. Everdur, Anaconda's group of copper-silicon alloys, also possesses high physical strength and resistance to wear and abrasion — so that wrought equipment can be designed with lighter weight.

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Write for Publication E-11, "Everdur Copper-Silicon Alloys for Sewage and Waterworks Equipment"—or for assistance of the Technical Department in selecting the correct material for your equipment. Address: The American Brass Company, Waterbury 20, Conn. In Canada: Anaconda American Brass Ltd., New Toronto, Ont.

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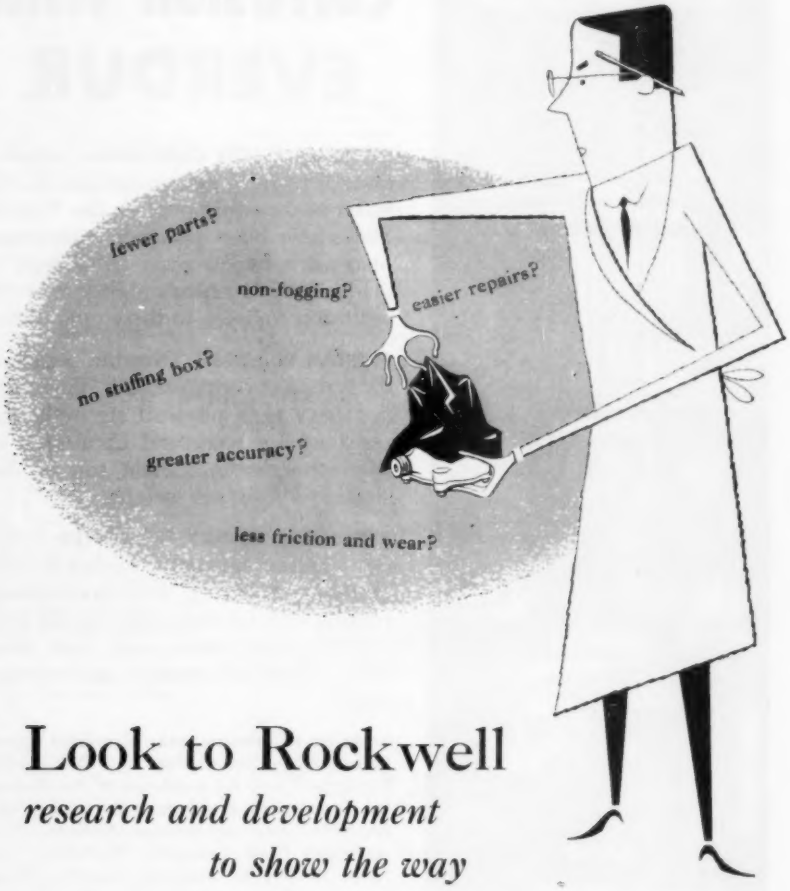

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Water Treatment, too, needs

INDIVIDUAL DIAGNOSIS

No two water treatment problems are exactly alike. The right solution to each can only be arrived at after a careful study of the local conditions. Variables such as raw water composition, rate of flow and results required automatically rule out the cure-all approach. The installation shown below is a good example of how equipment should be selected to fit the job . . . and not vice versa.

View of the Fort Lauderdale Fiveash Plant. Dorrco Hydro-Treator unit in foreground.
Consulting Engineers: Philpott, Ross & Saarinen, Fort Lauderdale, Florida.

Fort Lauderdale FLORIDA

Uses High-Rate Water Treatment to Meet Needs of Growing Population

Like every progressive city, in 1952 Fort Lauderdale was faced with expansion of water treatment facilities to meet the needs of its rapidly growing population. Careful analysis indicated that high-rate treatment would be the best answer to their problem.

The result was the new 8MGD Fiveash Water Plant which went into operation in December 1953 and has greatly reduced the load on the Old Dixie Plant. Finished water averages less than 50 ppm hardness with zero color. Treatment consists of aeration, softening, filtration and chlorination with a 55 ft. square Dorrco Hydro-Treator mechanism installed for the softening step.

If you'd like more information on the complete line of Dorr-Oliver water treatment equipment, write to Dorr-Oliver Incorporated, Stamford, Connecticut for Bulletin No. 9141.

Every day over 8½ billion gallons of water are treated in Dorr-Oliver equipment

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